

Canada - New Brunswick
Flood Damage Reduction Program

Hydrotechnical Study Of The Walker Brook Flood Plain

Restigouche County

Summary Report

January , 1983



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January 31, 1983

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Dear Mr. Lockhart:

Re: Walker Brook Hydrotechnical Study

We take pleasure in submitting our report on the above mentioned study. All comments and suggestions by the Hydrotechnical Committee from their review of the draft report have been incorporated in this version.

The report is presented in two parts - summary form followed by a technical discussion. The Summary Report follows this Letter of Transmittal and precedes the Main Report which more technically describes the study methodology and investigations.

We would like to express our sincere thanks to you and the other members of the Walker Brook Project Committee for your cooperation and assistance throughout the course of this study.

All of which is respectfully submitted.

Yours very truly,

SPEAR, MACDONALD & ASSOCIATES LTD.



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SUMMARY AND MAIN REPORTS

TO

CANADA-NEW BRUNSWICK

FLOOD DAMAGE REDUCTION PROGRAM

FOR A

HYDROTECHNICAL STUDY

OF THE

WALKER BROOK FLOOD PLAIN

by

Spear, MacDonald & Associates Limited, in association with

Cumming-Cockburn & Associates Limited,

January, 1983

HYDROTECHNICAL STUDY OF THE
WALKER BROOK FLOOD PLAIN

SUMMARY REPORT

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HYDROTECHNICAL STUDY OF THE
WALKER BROOK FLOOD PLAIN

SUMMARY REPORT

1. INTRODUCTION

1.1 The Flood Damage Reduction Program

A flood plain is the flat or gently sloping land adjacent to a stream which, during high streamflow conditions, caused by excessive rainfall or snowmelt or a combination of the two, becomes inundated by water. Many communities in Canada developed along rivers for a number of very good reasons. The nation's waterways were not only the major transportation arteries for the early settlers but, in many instances, served as a source of livelihood. As communities grew, so did occupation of the flood plain to the point where substantial damages, human suffering and indeed, loss of lives occurred during major floods.

During the 1970's, flood damages in Canada were approximately \$200 million, of which approximately 10% were incurred in New Brunswick. As flood damages escalated so did pressure to construct flood control works. Significant investments were made in structural works during the 1950's and 1960's. While structural works have been successful in many communities, this approach is not always wise. Structural works are costly to construct and maintain and can only be built to provide a finite level of protection. Because structural works are usually perceived to provide a higher, if not an absolute, level of protection than they are designed for, they usually attract more development on flood plain lands. In some situations, because structures cannot provide absolute protection against all floods, they can be an open invitation to a major catastrophe.

During the mid-1970's, the Federal Government developed the National Flood Damage Reduction Program. The basic objective of the program is to reduce flood damages through a comprehensive analysis of all feasible structural and non-structural approaches. The Province of New Brunswick was the first provincial government to enter into an agreement with the Federal Government (currently agreements have been entered into with 7 provincial governments). A Flood Risk Mapping Agreement and a Studies Agreement were signed in 1976 and a Flood Forecasting Agreement, to provide an effective flood warning system for the Saint John River Basin, was signed in 1977. All agreements were extended in 1981 for an additional 5 years. Agreements were also signed to implement remedial works in the Marsh Creek watershed. The City of Saint John was an equal partner with respect to this agreement. The Marsh Creek Agreement is a classical example of the new policy in that both non-structural and structural works are key elements of the basic approach to reduce flood damages.

The Flood Risk Mapping Agreement is the cornerstone of the Flood Damage Reduction Program. Under this Agreement, the flood risk areas along a number of watercourses in New Brunswick are being identified. This information is made available to the general public, to federal, provincial and municipal government agencies and to as many potential investors as possible. Usually, the flood risk map identifies two zones, the floodway and the floodway fringe. In New Brunswick, the floodway is defined as that area flooded on an average of once in twenty years or where there is a 5% chance of flooding in any given year. The floodway fringe makes up the remainder of the flood risk area. The floodway fringe includes all that area flooded on an average of once in one hundred years or where there is a 1% chance of flooding in any given year. Once flood risk maps have been prepared for an area, the federal and

provincial governments, through a formal designation process, agree not to engage in or fund developments in the floodway and will only participate in funding developments in the floodway fringe provided they are adequately floodproofed. Undertakings for agricultural and recreational purposes are excluded from this policy. Furthermore, once an area has been designated, the governments may withhold disaster assistance payments to developments constructed after designation has been effected.

The Studies Agreement provides for the evaluation of remedial measures to minimize flood damage potential in areas which have a significant flooding problem. Under the agreement, all feasible structural and non-structural measures must be evaluated in terms of effectiveness, cost, benefit, and environmental impact. Structural works include, but are not limited to, flood control dams, dykes, channel improvements and diversions. Non-structural measures include, but are not limited to, floodproofing, zoning, flood warning, emergency measures, land acquisition and watershed management.

1.2 Description of Walker Brook Basin

Walker Brook, located in Restigouche County in the northern part of New Brunswick, is a relatively small stream having a drainage area of about 67 square kilometres to its outlet. Walker Brook flows through the community of Val d'Amour, part of the Village of Atholville and through the City of Campbellton. The stream originates in the highlands south of Val d'Amour and flows northerly along a steep channel to discharge into the Restigouche River at Campbellton. The watershed and study area are shown on Figure 1.

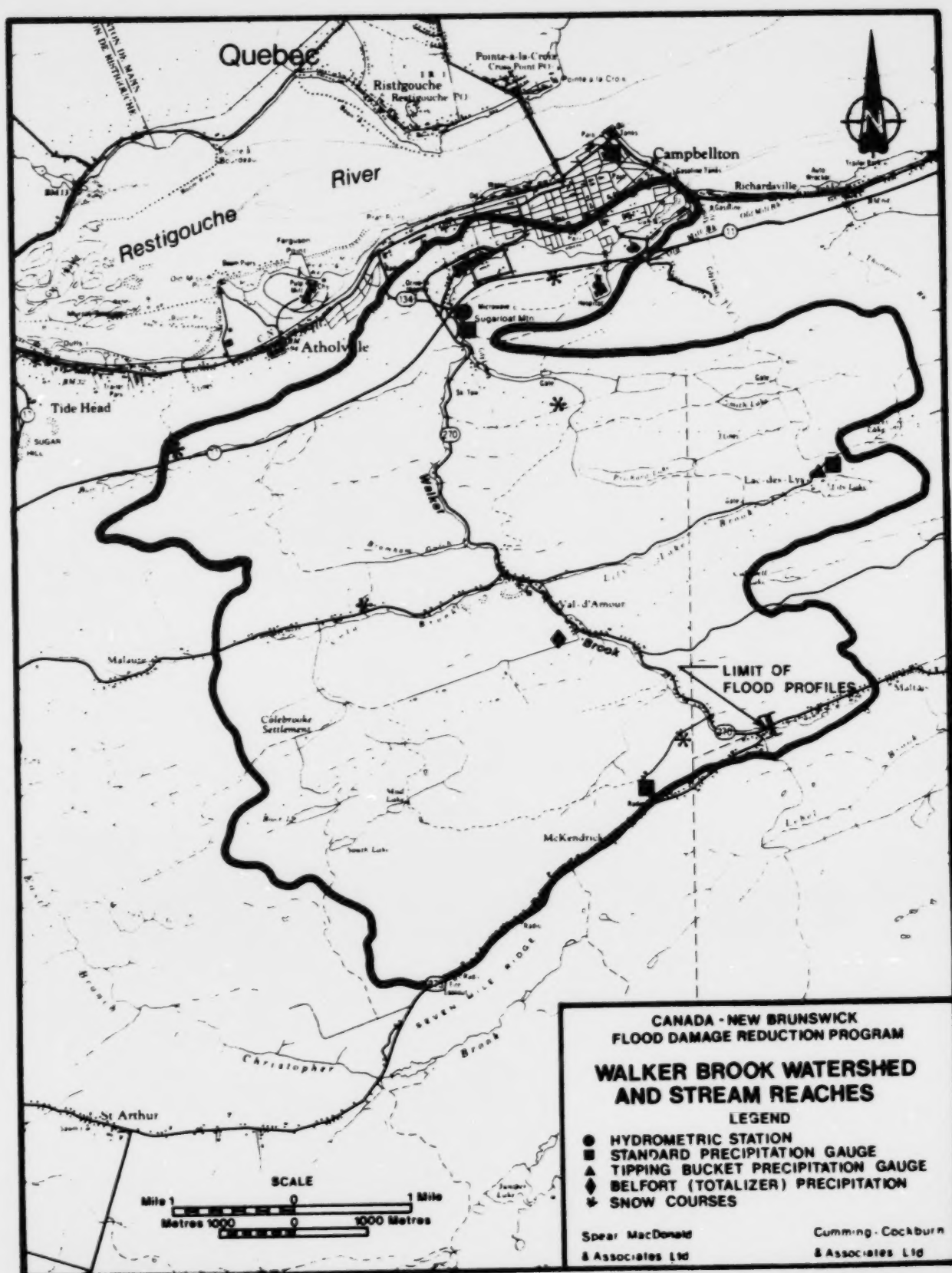


FIGURE 1

Two areas along Walker Brook have been identified as having a high flood potential. One major flood plain is between the Sugarloaf Provincial Park and the confluence of Walker Brook with the Restigouche River. The other area is along the river reach extending through the community of Val d'Amour.

There has been extensive development of the Walker Brook flood plain in recent years in the form of single family homes, apartments, low rental units and mobile homes. Additionally a large shopping mall, municipal offices and garage and several business establishments have been located near the stream and provincial tourist park facilities have been constructed.

The trend toward greater development is also apparent as several large tracts of land have been staked out for subdivision and reclamation of interval land is being accomplished by land filling. Plans have also been made to relocate principal access routes along and across the stream and this will require infilling. With this almost continuous development, a number of structures have been installed to permit travel across the brook and some of these are hydraulically underdesigned, creating impediments to flood flows and increased flooding.

There has also been considerable development in the Val d'Amour area. This is largely residential with homes being built in close proximity to the stream. In many cases, these locations appear to be prone to flood damage under high water conditions. Numerous small bridges have been built across Walker Brook in the Val d'Amour area.

1.3 Historical Flooding Along Walker Brook

Information on historical floods along Walker Brook covers the period 1928 to 1982. The earliest records suggest that flooding

of structures usually occurred in the vicinity of Val d'Amour Road and Alexander Street in Campbellton (see Figure 2); this area continues to suffer during flood conditions. Floods causing the most damage have taken place during the spring runoff periods, the most recent occurring during the spring of 1979.

Development of land in the Walker Brook flood plain has taken place mainly during the past ten years. Long term residents of the area believe that flood damages have increased in recent years corresponding to the continued development of flood plain lands.

The worst flood recorded in the area occurred in 1973. Towards the end of April of that year two days of heavy rain (about 100 to 130 mm) combined with snowmelt conditions caused the worst flood experienced during the past 50 years.

The following summary includes some examples of documented flooding and damage during the last decade (Environment Canada, 1976; Folster, 1980):

- In 1970 - Culvert washed out in St. Albert area and minor property damage
- In 1973 - Bridges washed out including Val d'Amour and Malauze Road.
 - Erosion on Val d'Amour Road
 - St. Albert area flooded, two small bridges washed out and several houses damaged
 - Gallant Drive bridge abutments eroded, road closed and ten families evacuated.
- In 1974 - Walkway in Val d'Amour washed out
 - Culvert under Route 270 west of Sugarloaf Park damaged
 - Water over road (about 25 centimetres deep) at Route 11 underpass

- Val d'Amour Road area residences damaged
- St. Albert culvert washed out
- Val d'Amour Road and Pine Street intersections flooded to a depth of one metre.

In 1976 - "Huge" lake formed on Water Street with a depth of about one-third of a metre.

In 1979 - Private stream crossings washed out between Val d'Amour and the City of Campbellton

- \$12,000 damage to Val d'Amour residents and local property erosion

- At Gallant Drive, the flood water flow was about 125 metres wide and one metre deep.

In 1980 - Private stream crossings washed out between Val d'Amour and the City of Campbellton.

Losses due to flooding have not been confined to property damage; over the past 50 years five children have lost their lives by drowning during flood conditions on Walker Brook.

1.4 The Walker Brook Project - A Phased Approach

The flood plain along Walker Brook has long been considered as one of the most flood prone areas in the Province and was high on the priority list in March, 1976 when the General Agreement Respecting Flood Damage Reduction and the Flood Risk Mapping Agreement were signed. It was also specifically mentioned in the Agreement Respecting Studies for Flood Damage Reduction. The continued development of flood prone lands has led to a corresponding demand for an increased level of protection from flood damages.

Consequently, a five phase project to help resolve the problems encountered in the past and to provide the technical basis for a flood plain management plan was proposed. A study to provide

information which will be useful in planning or implementing schemes for reducing flood damages along Walker Brook from Val d'Amour to the confluence with the Restigouche River was the first phase. This study is the subject of this summary report. The principle activities included in the phased approach are:

- Phase I - Hydrotechnical Study to include an analysis of the climatological, hydrological and hydraulic characteristics of the Walker Brook Basin.
- Phase II - Planning to determine stage-damage relationships and to identify feasible structural and non-structural flood damage reduction alternatives.
- Phase III - Evaluation of procedures for flood damage reduction measures including preliminary design, environmental impact analysis, preliminary cost estimates, benefit/cost analysis, flood risk mapping and public meetings.
- Phase IV - Design of flood damage reduction measures and detailed cost estimates.
- Phase V - Implementation to include construction of flood damage reduction measures.

The interdependence of the various phases requires that they be carried out sequentially with the participation and support of local residents in addition to governmental agencies. The need for undertaking each of the subsequent phases will depend on the findings of the immediately preceeding phase of the overall investigations.

1.5 Purpose and Scope of the Report

The purpose and scope of the hydrotechnical study were to provide the basic technical information that would enable schemes for reducing flood damages along Walker Brook to be planned and implemented. The investigations carried out during the course of the study included the:

- Review of available background data related to historical flood conditions
- Implementation of a field survey program
- Assessment of the climatologic and hydrologic and hydraulic characteristics of the Walker Brook Basin.
- Determination of water levels resulting from the once in 20 years and the once in 100 years flooding conditions and plotting the lines representing these elevations on topographical mapping.
- Assessment of ice and debris jamming, tides and other hydraulic factors affecting the flood lines
- Identification of possible remedial measures for flood management and flood damage reduction for investigation in subsequent phases of the program for Walker Brook.

A full description of the study has been published in a Main Report entitled "Hydrotechnical Study of the Walker Brook Flood Plain" and in a Supplementary Report to the above describing the field investigations, January, 1983.

2. THE WALKER BROOK HYDROTECHNICAL STUDY

2.1 Background Data

A number of earlier studies related to the Walker Brook watershed are listed in the reference section appended to this report. The report entitled "Regional Flood Frequency Analysis" (Acres, 1977) provides estimates of peak flows at various

locations along the Walker Brook. The report entitled "Climatological Data for the Walker Brook Basin in Relation to Flood Studies" (Environment Canada, 1981) provides an analysis of peak rainfall and snowmelt conditions in the basin. It was noted that the Campbellton area experiences one of the highest snowfall accumulations in Canada and that floods could result from both spring snowmelt and summer rainfall conditions.

Preliminary field surveys of the channel and flood plain characteristics were carried out in the study area by New Brunswick Environment (King, 1978) during which some cross-sections, channel profile and structures were measured.

Streamflow data have been collected by the Water Survey of Canada, Inland Waters Directorate, Environment Canada at a hydrometric station near Sugarloaf Provincial Park since November 1980. Additional streamflow data in the vicinity of the study area are available on the Eel River near Eel River Crossing.

Climatic records are obtained by field observers at several locations within the study area operated according to national standards under the direction of the New Brunswick Department of the Environment. Precipitation data have been observed on a daily basis at Seven Mile Ridge, Sugarloaf Mountain, Lily Lake, and Campbellton since June, 1981. Rate of rainfall data have also been observed at Lily Lake seasonally since 1981. In the vicinity of the study area, climatic records are also available at Balmoral, Charlo Airport and Tide Head.

Snow data have been collected by the New Brunswick Department of the Environment at five snow courses within the study area since the 1981 winter season.

Tidal information is published under the authority of the Canadian Hydrographic Service, Department of Fisheries and Oceans. Campbellton is a secondary tidal port where tidal information is given in relation to the predictions for the reference port "Pointe-Saint-Pierre" on the coast of the Gaspé Peninsula.

2.2 Field Program

The field program carried out as part of the hydrotechnical study included the installation of gauges in order to obtain water level measurements during peak run-off conditions, and the measurement of cross-sections and structure dimensions along the watercourse. Thirteen crest and fourteen staff gauges were strategically located in the main channel at culverts and bridge openings; they were installed and operational by April 24, 1982.

The physical surveys taken as part of the field program included defining structure types and sizes along the watercourse, obtaining representative cross-sections of the Walker Brook channel and flood plain and referencing these cross-sections to the New Brunswick Co-ordinate System.

The discharge and flood levels during peak flows along Walker Brook are controlled in part by some 32 bridges and culverts and four weirs, from its outlet to the upstream study limit. Many of these structures create a severe constriction to the watercourse and are frequently surcharged, thus increasing the flood risk to residences in the adjacent flood plain. However, a number of these structures along the upstream reaches are private stream crossings which could easily be washed out during the occurrence of peak flow events. Physical dimensions and control elevations of all structures were obtained during the field survey.

A total of 43 cross-sections were surveyed along Walker Brook from the confluence with the Restigouche River to the intersection of Highways 270 and 275 upstream of Val d'Amour near McKendrick. The cross-sections were located based on changes in the slope, cross-sectional area or channel roughness. Additional cross-sections were measured near bridge and culvert crossings.

A comparison of the 1978 and 1982 survey programs revealed changes in the cross-sectional elements in some areas due to the effects of erosion and sedimentation processes along the watercourse. For example, about 1 to 1.5 metres of sand and gravel have accumulated along the channel reach downstream of Dover Street. This is consistent to some extent with the observation of local residents that significant accumulations of sediment have severely restricted the hydraulic capacity of the Gallant Drive bridge. Apparently a significant amount of sand and gravel has accumulated to raise the bed of the channel at this location.

The analysis of background data and survey information revealed that two main reaches characterize the Walker Brook channel: upstream and downstream of Highway 11 near Sugarloaf Provincial Park. Upstream of Sugarloaf the channel is in a more natural state and is winding and fairly steep following Route 270 through Val d'Amour. The channel banks are relatively clean and well defined while the channel bed is comprised of stones and cobbles. There is little channel vegetation present due to the high flow velocities and steep bedslope associated with the watercourse, except for localized areas upstream of Val d'Amour and near the snowmobile crossing to Sugarloaf.

Downstream of Sugarloaf the man-made influences on the Walker Brook channel are more diverse, with extensive areas of rip-rap protection. Some areas of bank erosion and sediment deposition in the lower portion of the watercourse are evident.

Similarly, the characteristics of Walker Brook flood plain also differ substantially upstream and downstream of the Sugarloaf - Highway 11 area. Upstream of Sugarloaf the overbank areas are heavily vegetated with trees and brush in most areas. The width of the flood plain is relatively narrow due to the steep valley confining the watercourse. In general, the eastern overbank has a relatively higher potential flow capacity since the channel follows Route 270. However, the western overbank is generally a mixture of forest and residential usage which would tend to constrict the passage of flow.

2.3 Hydrology

The area inundated by a particular flood depends on the magnitude of the peak flow which in turn depends on the prevailing climatologic conditions in the watershed. Considerable investigation is necessary to develop reasonable peak flow estimates using the best techniques available. For the purpose of this study, alternative estimating methods were developed and compared;

- i) statistical estimates of peak flows using available streamflow records
- ii) computer simulation of peak flows using climatologic records.

The use of alternative techniques was undertaken in order to provide reliable estimates of the 1:20 and 1:100 year return period flood flows.

i) Statistical Estimates

Usually floods to be mapped are determined by frequency analyses based on the assumption that floods are natural events that can be described by a particular probability distribution. The frequency analysis involves obtaining and assessing all the recorded information, checking for the possibility of ice or log jams and using the appropriate theoretical probability distribution technique.

Single site flood frequency analysis for the watercourse being studied is adequate only if the record is long and reliable. These criteria could not be met, since the data available from the Walker Brook hydrometric station are not of sufficient length for a reliable statistical analysis. Therefore a regional flood frequency analysis was utilized (Acres 1977) in which case several single site analyses were combined by using data from hydrometric stations in the region.

The transfer of peak flow estimates from another watercourse, close to the study area and having sufficient and reliable recorded data, was also used to estimate flood magnitudes. Our investigations revealed that it was possible to transfer peak flow estimates from the Eel River to the Walker Brook Watershed in order to provide additional confirmation of peak flow estimates.

ii) Computer Simulations

Climatological records, including historical regional storms and snowmelt combined with precipitation, were also used as a basis for estimating flood peaks. The runoff produced by these conditions and the discharge of the watercourse were computed using an appropriate hydrologic model of the watershed.

In this case, a computer technique called "HYMO" was used to model peak flows. The model takes into account all the factors which have a major influence on the runoff characteristics and was tested by comparison of recorded and simulated flood flows for historical events along Walker Brook.

The hydrologic component of the study included ascertaining the response of the Walker Brook watershed to major climatic events such as rain storms, rapid snowmelt or a combination of both. In addition to computer simulation of 1:20 and 1:100 year return period open water flood flows, peak flows associated with estimates of probable maximum precipitation, as determined by the atmospheric Environment Service, Environment Canada, and maximum precipitation associated with Hurricane Belle, transposed over the watershed, were estimated.

The comparison of statistical estimates and the computer model results confirmed that the flood peaks estimated by the Regional Flood Frequency Analysis are reasonable and reliable values which are appropriate for use in the Walker Brook watershed.

The following table summarizes the peak flows estimated for Walker Brook at several locations.

<u>Location</u>	Peak Flows (in cubic metres per second)	
	Once in 20 Years	Once in 100 years
(a) Walker Brook near outlet	57.9	82.9
(b) Walker Brook near Sugarloaf Mountain	53.4	76.5
(c) Walker Brook upstream from Sugarloaf	42.3	61.1

Had the maximum precipitation associated with Hurricane Belle fallen over the watershed, it has been estimated that the resulting peak flows would have been approximately 100 cubic metres per second. Furthermore, it was estimated that the peak flow associated with probable maximum precipitation estimates would exceed 350 cubic metres per second. It should be born in mind that this latter estimate by definition would be an extremely rare occurrence and would likely result in devastating flood damages.

2.4 Hydraulics

Computation of water surface profiles for the 1:20 and 1:100 year flood events was undertaken by using a computer program referred to as the HEC-2 backwater model. The model takes into account a number of hydraulic factors describing the channel and flood plain along the study reach and is one of the most widely used and accepted techniques currently available for computing flood profiles.

Input parameters were varied along the study reach according to the findings of the field surveys as discussed in section 2.2 and the findings of the hydrologic analysis as discussed in section 2.3. The HEC-2 model can be used in the future to evaluate the effects of future changes including recommended hydraulic improvements and proposed channelization along the study reach.

Unfortunately it was not possible to compare computed and observed flood profiles along the study reach since suitable flood conditions were not observed during the field program.

An analysis of discharge and flood conditions associated with the 1:20 and 1:100 year flood peaks revealed that during peak flow

events, a significant amount of spill from the channel would occur at the following locations (Figure 2):

- Old dam downstream from Sugarloaf Park (flows under the Highway 11 overpass and partially rejoins the main channel in the vicinity of the Municipal garage; the balance flows along Val d'Amour Road diversion and rejoining the main channel near Alexander Street).
- Overpass of Highway 11 bridge at Route 270 (flows in ditch along Highway 11 rejoining the main channel near Gallant Drive).

Flow spilled from the channel and flood plain at these locations tends to reduce the peak flows and flood levels at locations immediately downstream. However, the flood levels are again increased at the point where the spilled flow rejoins the main channel of Walker Brook. Rating curves were developed for each spill location in order to calculate the amount of flow spilled and the effect on the downstream flood levels.

The hydraulic analyses also included consideration of the potential for ice jam formation. It was found that the largest volume of ice is apparently produced each year below the Water Survey of Canada gauge located near Sugarloaf Mountain. Hence, a larger potential for ice jams exists along the lower portion of the study area. As a result of these investigations, it was found that significant ice and debris jams occur mainly at those locations along the channel where man-made obstructions to the flow are present.

The most significant effect of ice and debris jams was found to occur at Village Road and Gallant Drive and at the curve in the channel just upstream of Gerard Street. Complete blockage of the

flow conveyance structures could occur at these locations, resulting in discharge over the road. However, the effect on upstream backwater levels would be fairly localized at both these locations due to the relatively steep channel profiles.

2.5 Discussion of Results

Typical channel and flood plain characteristics for various reaches were determined for Walker Brook. Generally speaking, the channel can be described according to two main reaches, namely that area upstream of Highway 11 near Sugarloaf Provincial Park through to Val d'Amour and that area downstream through the City of Campbellton. The downstream areas are characterized by extensive residential and commercial development located on a relatively wide flood plain. The damage potential to structures in this area is envisaged to be quite high. In the upstream area, the channel is narrow and fairly deep and the flood hazard is confined mainly to a number of homes and small bridges which have been constructed to cross the channel, particularly in the Val d'Amour area.

From the confluence of the Brook with the Restigouche River up to Village Road, the flood hazard was found to be relatively low since the 1:20 and 1:100 year flood lines do not encroach on any existing urban developments.

Upstream of Village Road to Dover Street the flood hazard is higher since overtopping of both Village Road and Gallant Drive occurs for both the 1:20 and 1:100 year events. The potential for ice and debris jamming could also cause discharge over the road if the flow conveyance structures at Village Road and Gallant Drive are completely blocked. However, few buildings are directly affected by flooding in this reach.

A moderate flood hazard exists in the reach from Dover Road to Pine Street. Considering that spillage occurs in the ditch on the south side of Highway 11, the flow in the Brook north of the Highway (spill adjusted) would be confined in the channel and flood plain while the 1:100 year unadjusted flow (no spill into Highway ditch) would inundate some structures in the vicinity of Pine and Ila Streets. From Pine Street up to the area bounded by Highway 11 and Val d'Amour Road extension (the extent of available mapping) the flood hazard is very high. The extent of the unadjusted 1:100 year floodline would inundate approximately 160 structures. On the other hand, if the ditch along Highway 11 acts as an overflow, then the flood hazard is significantly reduced and about 40 structures would be affected along this reach.

Upstream of Highway 11, the peak flows are generally contained in the natural channel and flood plain, with the exception of those spill areas previously mentioned. The main flooding hazards along the upstream channel reaches appear to be related to high discharge velocities and to the potential for washout of several wooden access bridges at various locations along the study area.

3. POSSIBLE REMEDIAL MEASURES

Having identified the specific flood prone areas along Walker Brook, as summarized in the previous section of this report, it was possible to consider alternative remedial measures for alleviating flooding problems.

In general, the basic elements for a flood damage reduction plan can be classified as:

- Non-structural measures which are intended to modify potential losses by discouraging new development in flood prone lands and encouraging flood proofing of existing structures.
- Structural measures which change the characteristics which contribute to the flooding problem.

A detailed analysis of potential remedial measures was beyond the scope of the present study; this will form part of the second and third phases of the Walker Brook project. However,

based on the results of this study, possible flood control measures discussed in the following sections are suggested for future detailed consideration.

3.1 Non-Structural Alternatives

To reduce the possibility of increased flood damages in the future, flood plain regulations could be drafted and imposed to control development in flood hazard areas along Walker Brook. The regulations could take the form of a two-zone concept; a high hazard floodway zone and a floodway fringe zone. In the high hazard areas of the floodway, construction of new development would be prohibited. In the floodway fringe areas some new development of the land may be allowed depending upon the degree of hazard and the provision of approved flood proofing methods to protect the new development from future flood damage.

Other measures to reduce claims for flood damages could involve the relocation of undertakings presently in the high hazard zone or implementing a funding program to help property owners to flood proof their lands or structures. The suitability of these and other measures would have to take into account the cost effectiveness, benefit, environmental and social concerns.

3.2 Structural Alternatives

The following structural control measures, in order of priority, could be considered along Walker Brook:

1. Flow diversion at Highway 11. The existing diversion of flow from Walker Brook along Highway 11 could be investigated further to define the feasibility of designing a permanent flow diversion structure.
2. Removal of flow obstructions. This could include removal of obstructions to flow at the following locations:
 - (a) Removal of abandoned dam near Sugarloaf Mountain
 - (b) Replacement of the culverts at Village Road (if cost/benefit analysis proves this to be viable)

- (c) Replacement of the bridge crossing at Gallant Drive
 - (d) Removal of the abandoned dam downstream from Village Road.
3. Construction of dykes or flood berms along selected reaches of the channel. The feasibility of constructing a flood control berm along both sides of the channel immediately downstream of Highway 11 could be considered. This would protect the flood plain areas between Highway 11 and Dover Street, and will require twinning or enlargement of the Dover Street culvert.
4. Additional stream channel improvements could be considered including:
- (a) Clearing of vegetation from the channel in the area downstream of Village Road
 - (b) Dredging of gravel and cobbles from the channel above Gallant Drive downstream to Village Road
 - (c) Straightening portions of the channel downstream of Dover Street to Gallant Drive particularly in the vicinity of Gerrard Street.
 - (d) Channel enlargement (for example, in the area from just downstream of Highway 11 to just downstream of Dover Street).

The most attractive structural alternatives presently appear to be the diversion of some flow along Highway 11, together with the replacement of the culverts at Village Road and the bridge at Gallant Drive. The latter remedial measures would also reduce the potential for ice and debris jams at these locations.

Gallant Drive is the main access to the provincial hospitals from the Campbellton side of Highway 11. A possible road

realignment between the floodway and Highway 11 is being considered. Accordingly, the feasibility of replacing the Gallant Drive bridge should take into account the proposed road realignment that may avoid the part of Gallant Drive susceptible to flooding.

If the flow diversion along Highway 11 proves to be feasible, minor culvert improvements should be carried out at Pine and 11a Streets, Alexander Street and Dover Street.

4. CONCLUSIONS AND RECOMMENDATIONS

The flood plain along Walker Brook has long been considered as one of the most flood prone areas in New Brunswick. The continued development in the area has aggravated the flooding problem. The flood hazard is highest between Sugarloaf Provincial Park and Village Road and in particular between Dover and Pine Streets. In addition there are a number of undersized stream crossings in the Val d'Amour area which can and have been taken out during flooding conditions.

Floods causing the most damage are reported to occur during the spring runoff periods. However, the spring of 1982, during which the field survey program for the study was carried out, did not produce any significant flows in the watercourse nor did any flooding occur within the study area.

In order to finalize calibration of the hydrologic model and to calibrate the backwater model it is necessary to use flooding events that produce flows of the same magnitude as the 1:20 and 1:100 year design events. It is recommended therefore that the collection and recording of meteorological, water levels, discharge, ice jam formation and ice thickness data be continued for as long as possible so that future calculations can be verified for more severe flood conditions.

Detailed topographical mapping is required to accurately define the nature and extent of the design flood events within the flood plain. The 1:1200 scale mapping available for the study extends to just upstream of Dover Street near the Campbellton City limits. Accordingly, it is recommended that up-to-date, topographical mapping be obtained for the flood prone lands along Walker Brook for the purpose stated; this mapping will also form the basis for the eventual production of the flood risk maps. In addition, a study of the surface drainage patterns in the St. Albert area should be carried out to accurately define the extent of potential flooding and possible relief measures.

When all the agencies involved in the Walker Brook project have reviewed the findings of the first phase it is recommended that the next phase of the program be carried out as soon as possible. This should include a field inventory to identify and classify structures subject to flood hazard and the determination of costs and benefits of a flood control program. In the meantime, authorities having control of development along Walker Brook should be advised of the flood hazard areas identified in the study so that they may discourage the construction of new works which may be subject to flood damages.

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ÉTUDE HYDROTECHNIQUE DE LA
PLAINE INONDABLE DE WALKER BROOK

RÉSUMÉ DU RAPPORT

JANVIER 1983

Canada - Nouveau-Brunswick
Programme de réduction des dommages
causés par les inondations

Étude hydrotechnique de
la plaine inondable de
Walker Brook

Comté de Restigouche

résumé du rapport

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ÉTUDE HYDROTECHNIQUE DE LA PLAINE

INONDABLE DE WALKER BROOK

RÉSUMÉ DU RAPPORT

1. INTRODUCTION

1.1 Programme de réduction des dommages causés par les inondations

Une plaine inondable est un terrain plat ou en pente faible situé près d'un cours d'eau qui l'envahit au moment de débits torrentiels engendrés par des précipitations excessives, ou par la fonte des neiges, ou par un ensemble de ces deux causes. Au Canada, beaucoup de localités se sont établies le long de rivières pour un certain nombre de très bonnes raisons. Le réseau fluvial du pays n'a pas représenté qu'un moyen de transport des premiers colons mais, dans beaucoup de cas, a servi de moyen d'existence. À mesure que s'implantaient les localités, les plaines inondables ont été occupées au point d'entraîner des dommages considérables, des souffrances humaines et même des pertes de vie au moment des grandes inondations.

Au cours des années 1970, les dommages causés par les inondations au Canada se sont chiffrés à environ deux cent millions de dollars, dont 10% au Nouveau-Brunswick. L'escalade des dommages dus aux inondations a fait naître les pressions visant à faire construire des ouvrages de protection contre les crues. On a engagé des sommes importantes dans des travaux de protection au cours des années 1950 et 1960. Si ces travaux se sont révélés efficaces dans beaucoup de localités, cette méthode n'est pas toujours pertinente. L'érection et l'entretien d'ouvrages de protection coûtent cher et ces installations ne fournissent qu'une protection limitée. Parce qu'on s' imagine que ces travaux fournissent une protection supérieure à celle prévue à la conception, sinon absolue, ils activent généralement la construction de bâtiments dans les plaines inondables.

ÉTUDE HYDROTECHNIQUE DE LA PLAINE
INONDABLE DE WALKER BROOK

RÉSUMÉ DU RAPPORT

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Ces ouvrages de protection, qui ne peuvent garantir une protection absolue contre toutes les inondations, peuvent provoquer la catastrophe.

Au milieu des années 1970, le gouvernement fédéral a instauré un programme de réduction des dommages causés par les inondations. Ce programme a pour premier objectif de réduire les dommages dus aux inondations à la suite d'une analyse globale de toutes les solutions possibles, soit la mise en place d'ouvrages de protection, soit toute autre approche au problème. La province du Nouveau-Brunswick a été la première administration provinciale à signer une entente avec le gouvernement fédéral (depuis, sept gouvernements provinciaux ont aussi conclu des ententes). En 1976, la province signait une entente concernant la cartographie des risques d'inondations et une entente portant sur des études et, en 1977, on concluait une entente concernant la prévision des inondations afin de doter le bassin de la rivière Saint-Jean d'un système avertisseur efficace des crues. En 1981, on a prolongé toutes ces ententes pour une durée de cinq ans. On a aussi signé des ententes pour aménager des ouvrages de protection dans le bassin hydrographique de Marsh Creek. La ville de Saint-Jean a aussi été partie contractante à cette entente. L'entente de Marsh Creek constitue une illustration classique de la nouvelle politique parce qu'on y fait appel tant aux travaux de protection qu'aux autres solutions comme éléments clés de la méthode fondamentale de réduction des dommages causés par les inondations.

L'entente concernant la cartographie des risques d'inondation représente la pierre angulaire du Programme de réduction des dommages causés par les inondations. Aux termes de cette entente, on détermine les zones qui présentent des risques d'inondation le long d'un certain nombre de cours d'eau du Nouveau-Brunswick. On met ces renseignements à la disposition du grand public, des organismes des administrations fédérale

et provinciale ainsi que des municipalités et au plus grand nombre de promoteurs possibles. Généralement, une carte du risque d'inondation détermine deux zones, le canal de crue et la zone périphérique. Au Nouveau-Brunswick, le canal de crue est la zone susceptible d'être inondée une fois tous les vingt ans, ou dont le risque d'inondation chaque année est de 5%. La zone périphérique est constituée du reste de la zone inondable. La zone périphérique englobe le secteur envahi par les inondations en moyenne une fois tous les cent ans, ou dont le risque d'inondation est de 1% par année. Une fois établies les cartes du risque d'inondation, les gouvernements fédéral et provincial, au moyen d'un processus formel de désignation, conviennent de ne pas prendre part à des projets de construction dans le canal de crue, ou d'y engager des crédits, et de n'affecter des fonds à des travaux de construction dans la zone périphérique que si ces bâtiments sont convenablement protégés contre les crues. De plus, une fois une zone désignée, les gouvernements peuvent refuser d'indemniser les victimes d'inondation s'ils ont érigé des bâtiments dans cette zone après la désignation.

L'entente portant sur des études vise l'évaluation des mesures correctives destinées à minimiser les risques d'inondation dans les secteurs fortement sujets aux crues. Aux termes de l'entente, on doit évaluer tous les ouvrages qu'il est possible d'ériger ainsi que toutes les autres solutions en terme d'efficacité, de rentabilité économique et d'incidence sur l'environnement. Dans les ouvrages, on comprend, entre autres, les barrages de régularisation des crues, les digues, les travaux d'amélioration du canal et les travaux de dérivation. Les mesures autres que les ouvrages incluent, entre autres, la protection contre les crues, le zonage, les systèmes avertisseurs d'inondation, les mesures d'urgence, l'achat de terrains et l'aménagement du bassin hydrographique.

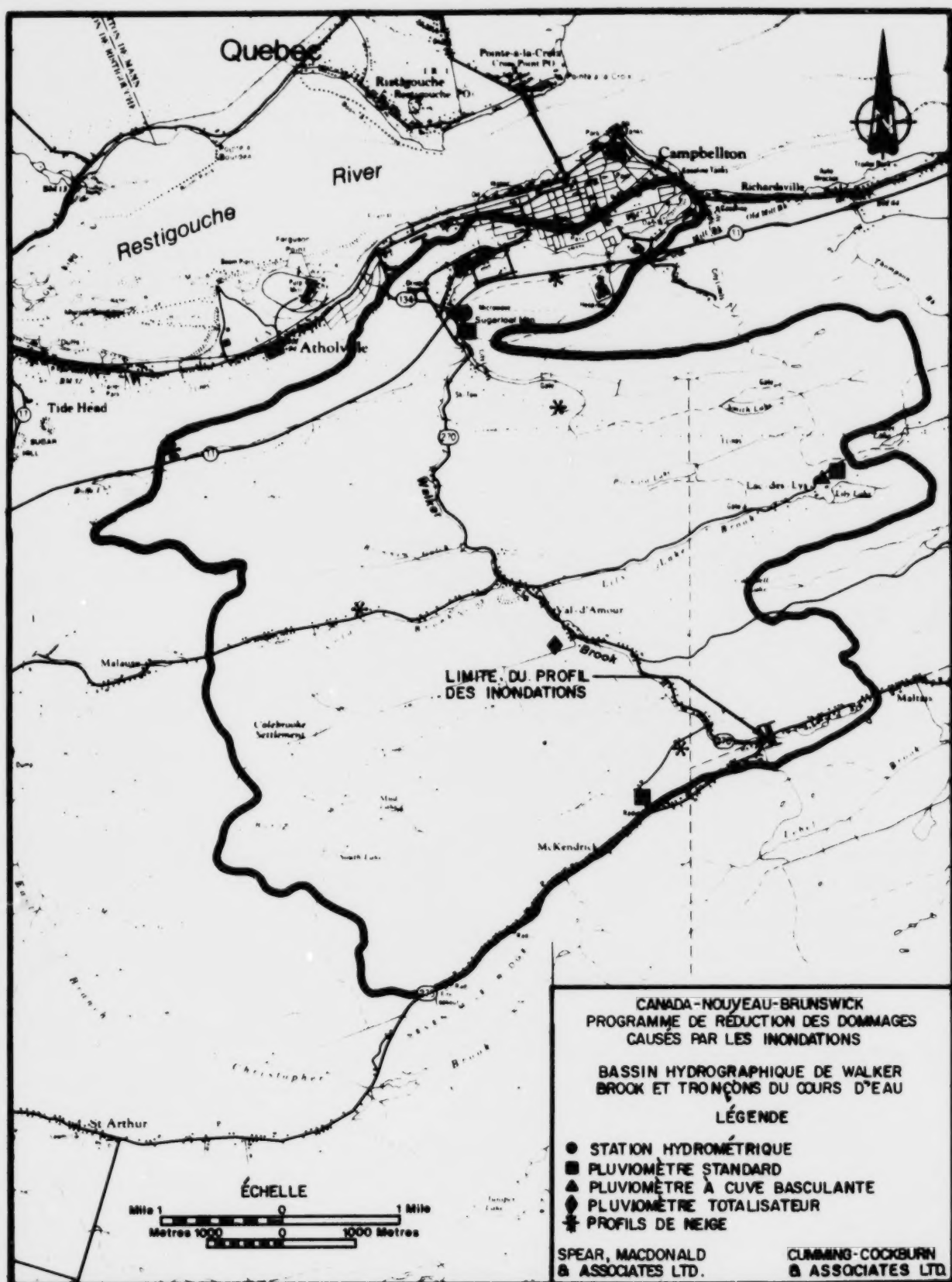


FIGURE 1

1.2 Description du bassin de Walker Brook

La rivière Walker Brook, qui est située dans le comté de Restigouche au nord du Nouveau-Brunswick, est un cours d'eau de faible importance qui s'alimente à une zone de drainage d'environ 67 kilomètres carrés jusqu'à sa sortie d'eau. Ce cours d'eau arrose la localité de Val d'Amour, une partie de la municipalité d'Atholville avant de traverser la ville de Campbellton. Il prend sa source dans les hautes terres situées au sud de Val d'Amour et coule vers le nord dans un canal en pente raide avant de se jeter dans la rivière Restigouche, à Campbellton. Le bassin hydrographique et la zone qui fait l'objet de l'étude sont illustrés à la figure 1.

On a relevé le long de Walker Brook deux zones qui présentent des risques élevés d'inondation. Une importante plaine inondable s'étend entre le parc provincial Sugarloaf jusqu'au confluent des rivières Walker Brook et Restigouche. L'autre plaine se trouve le long du tronçon du cours d'eau qui traverse la localité de Val d'Amour.

Au cours des dernières années, on a construit dans la plaine inondable de Walker Brook beaucoup d'habitations unifamiliales, d'immeubles à appartements, d'habitations à loyer modique ainsi que de maisons mobiles. De plus, on a aussi aménagé près du cours d'eau un grand centre commercial, un garage et des bureaux municipaux ainsi que plusieurs établissements commerciaux, outre les installations touristiques du parc provincial.

La tendance à l'aménagement de ce secteur se poursuit puisque plusieurs vastes étendues de terrains ont été jalonnées aux fins du lotissement et on empiète sur les parcelles intercalaires en y déversant du remblai. On prévoit également déplacer les principales voies d'accès qui longent et traversent le cours d'eau, ce qui nécessitera des travaux de remblai. Outre cette mise en valeur presque permanente des terrains, on y a aussi installé un certain nombre d'ouvrages qui enjambent le cours

d'eau et la conception de certains d'entre eux n'est pas conforme aux principes de l'hydraulique, ce qui altère l'écoulement des crues et accroît les risques d'inondation.

La zone de Val d'Amour est aussi soumise à un aménagement intensif. On y retrouve surtout des maisons construites très près du cours d'eau. Dans beaucoup de cas, ces habitations pourraient subir des dégâts advenant de fortes crues. Dans la zone de Val d'Amour, de nombreux petits ponts traversent la rivière Walker Brook.

1.3 Inondations historiques le long de Walker Brook

Les renseignements dont on dispose sur les inondations mémorables survenues le long de Walker Brook couvrent la période allant de 1928 à 1982. Les archives les plus anciennes laissent entendre que l'inondation de bâtiments survenait généralement dans les environs de la route de Val d'Amour et de la rue Alexander à Campbellton (voir figure 2); ce secteur est toujours sujet aux inondations. Les crues les plus dommageables se produisent au cours de la fonte des neiges du printemps, la dernière survenant au printemps de 1979.

C'est principalement au cours des dix dernières années qu'on a mis en valeur les terres de la plaine inondable de Walker Brook. Des résidents de longue date de la zone croient que les dommages causés par les inondations s'accroissent depuis quelques années en raison de l'aménagement des plaines inondables.

La pire inondation enregistrée dans la région est survenue en 1973. Vers la fin d'avril de cette année, deux jours de pluies torrentielles (entre 100 et 130 mm) en plus de la fonte des neiges ont entraîné les pires crues connues dans la région depuis les cinquante dernières années. Vous trouverez ci-dessous certains exemples d'inondations qu'on a relevées et des dégâts qu'elles ont causés au cours de la dernière décennie (Environnement Canada, 1976, Folster, 1980):

En 1970 - Un ponceau emporté dans la zone de Saint-Albert et des dommages faibles à la propriété.

En 1973 - Des ponts emportés, y compris ceux des chemins de Val d'Amour et Malauze

- Érosion du chemin de Val d'Amour
- Inondation de la zone de Saint-Albert, deux petits ponts emportés et plusieurs maisons endommagées
- Les culées du pont de la promenade Gallant subissent l'érosion et dix familles sont évacuées.

En 1974 - La passerelle de Val d'Amour s'effondre à cause des crues

- Un ponceau de la route 270, à l'ouest du parc Sugarloaf, est endommagé
- La route recouverte d'environ 25 centimètres d'eau au niveau du passage inférieur de la route 11
- Des résidences de la zone du chemin de Val d'Amour sont endommagées
- Le ponceau de Saint-Albert emporté par les crues
- Les intersections du chemin de Val d'Amour et de la rue Pine enfouies sous un mètre d'eau.

En 1976 - Un "énorme" lac d'une profondeur d'environ 33 centimètres se forme sur la rue Water.

En 1979 - Les traversées privées qui enjambent le cours d'eau entre Val d'Amour et Campbellton emportées par les crues

- Des résidents de Val d'Amour enregistrent des pertes de 12 000 \$ à cause des dommages à leur propriété et de l'érosion
- À la promenade Gallant, les crues recouvrent une superficie de 125 mètres de largeur et d'un mètre de profondeur.

En 1980 - Des traversées privées du cours d'eau entre Val d'Amour et Campbellton emportées.

Les pertes dues aux crues ne se limitent pas aux dommages à la propriété; au cours des cinquante dernières années, cinq enfants se sont noyés au cours d'inondations de la rivière Walker Brook.

1.4 Le projet de Walker Brook - Une approche par étapes

Depuis longtemps, on a considéré que la plaine inondable de Walker Brook constituait une des zones de la province les plus sujettes aux crues et, en mars 1976, on l'a inscrite parmi les premières priorités au moment de signer l'entente cadre concernant la réduction des dommages causés par les inondations et l'entente concernant la cartographie des risques d'inondation. Cette zone est aussi clairement mentionnée dans l'entente portant sur les études visant la réduction des dommages dus aux inondations. La poursuite de l'aménagement des terres sujettes aux inondations a entraîné la nécessité d'augmenter la qualité de la protection contre les crues.

On a donc proposé un projet en cinq volets destiné à contribuer à résoudre les problèmes qu'on a connus par le passé et à fournir des fondements techniques à un plan de gestion de la plaine inondable. La première étape consistait à mener une étude visant à recueillir des données nécessaires à l'élaboration et à la mise en oeuvre de plans destinés à réduire les dommages occasionnés par les crues de Walker Brook entre Val d'Amour et le confluent de la rivière Restigouche. C'est cette étude qui fait l'objet du présent rapport. Voici quels sont les principaux volets de ce projet:

- Volet I - L'étude hydrotechnique qui analyse les caractéristiques climatologiques, hydrologiques et hydrauliques du bassin de Walker Brook.
- Volet II - La planification qui vise à établir le rapport entre les dommages et le niveau d'inondation ainsi qu'à déterminer les méthodes de réduction des dommages dus aux inondations telles que la construction d'ouvrages de protection ou toute autre solution.
- Volet III - L'évaluation des modalités des mesures visant à réduire les dommages causés par les crues, y compris l'élaboration d'un avant-projet, l'analyse de leur incidence sur l'environnement, l'établissement des coûts estimatifs, l'analyse coûts-avantages, la cartographie des risques d'inondation et les audiences publiques.
- Volet IV - L'élaboration de mesures destinées à réduire les dégâts occasionnés par les crues et l'établissement des coûts détaillés.
- Volet V - La mise en oeuvre des mesures de réduction des dommages causés par les inondations.

L'interdépendance de ces diverses étapes exige qu'on les mette en oeuvre dans un ordre séquentiel et avec la participation et le soutien des résidents de la localité ainsi qu'avec le concours des organismes de l'État. La nécessité d'entreprendre l'application des solutions prévues à chaque volet dépend des résultats obtenus à l'étape précédente.

1.5 But et portée du rapport

L'étude hydrotechnique avait pour objectif de fournir les données techniques essentielles à l'élaboration et à l'application des solutions permettant de réduire les dommages causés par les inondations le long de la rivière Walker Brook. Les recherches réalisées au cours de l'étude comprenaient:

- la revue des données historiques disponibles en ce qui concerne les conditions des inondations;
- l'application d'un programme de relevés topographiques;
- l'évaluation des caractéristiques climatologiques, hydrologiques et hydrauliques du bassin de Walker Brook;
- l'établissement des niveaux d'inondation résultant des conditions propres aux crues à tous les 20 ans et tous les 100 ans et transposer le tracé de ces élévations sur une carte topographique;
- l'évaluation des embâcles dues aux glaces et aux débris, des marées ainsi que des autres facteurs qui influent sur la hauteur des crues;
- l'élaboration de mesures correctives possibles en ce qui a trait aux crues et à la réduction des dommages qu'entraînent ces crues, solutions qui seront approfondies dans les étapes ultérieures du programme de Walker Brook.

On peut trouver une description complète de l'étude dans un rapport publié en janvier 1983 et intitulé "Hydrotechnical Study of the Walker Brook Flood Plain" et dans un rapport complémentaire aux données présentées ci-dessus et qui décrit les recherches réalisées sur le terrain.

2. L'ÉTUDE HYDROTECHNIQUE DE WALKER BROOK

2.1 Données de base

La liste des références annexée au présent rapport porte sur les études réalisées précédemment et touchant le bassin hydrographique de Walker Brook. Le rapport intitulé "Regional Flood Frequency Analysis" (Acres 1977) fournit une estimation des débits de pointe obtenus à divers emplacements de la rivière Walker Brook. Le rapport dont le titre est "Données climatologiques du bassin de Walker Brook en rapport avec les études portant sur les crues" (Environnement Canada, 1981) fait l'analyse des conditions qui prévalent dans le bassin au moment des précipitations de pointe et de la fonte des neiges. On a noté que la région de Campbellton connaît les plus grandes accumulations de neige au Canada et que les inondations peuvent résulter tant de la fonte des neiges au printemps que des pluies l'été.

Environnement Nouveau-Brunswick (King, 1978) a réalisé des relevés topographiques préliminaires touchant les caractéristiques du canal et de la plaine inondable de la zone en question. On a alors mesuré certaines coupes transversales, le profil et les structures du canal.

Depuis novembre 1980, Relevés hydrologiques du Canada, Direction générale des eaux intérieures, Environnement Canada, a recueilli des données concernant le débit du cours d'eau à une station hydrométrique située près du parc provincial Sugarloaf. On possède aussi des données relatives aux débits de la rivière Eel River, près de Eel River Crossing, qui se trouve près de la zone d'étude.

On obtient en outre des relevés climatiques grâce à des observateurs postés sur le terrain à plusieurs endroits à l'intérieur de la zone étudiée et oeuvrant selon les normes nationales et sous la direction du ministère de l'Environnement du Nouveau-Brunswick. Depuis juin 1981, on a pu procéder à des relevés quotidiens des précipitations à Seven Mile Ridge, Sugarloaf

Mountain, Lily Lake et à Campbellton. On fait à Lily Lake depuis juin 1981 des relevés relatifs au taux saisonnier des précipitations. On peut également disposer de relevés climatiques propres au secteur entourant la zone d'étude, c'est-à-dire à Balmoral, à l'aéroport de Charlo et à Tide Head.

Le ministère de l'Environnement du Nouveau-Brunswick a réalisé des relevés portant sur la neige à cinq lignes de relevés d'enneigement situées à l'intérieur de notre zone depuis l'hiver 1981. Le Service hydrographique du Canada, des Pêches et Océans, publie les renseignements concernant les marées. Campbellton constitue un port secondaire de diffusion de renseignements relatifs aux marées par rapport aux prévisions établies par le port de référence qu'est Pointe-Saint-Pierre situé sur la côte de la péninsule gaspésienne.

2.2 Relevés sur le terrain

Le programme de relevés sur le terrain exécutés dans le cadre de l'étude hydrotechnique comprend l'installation de jauges destinées à mesurer les niveaux d'eau au moment des ruissellements de pointe ainsi qu'à prendre des coupes transversales et des dimensions de la structure du canal le long du cours d'eau. On a installé, à certains ponceaux et aux orifices des ponts sur le canal principal, treize jauges de crête et quatorze échelles limnimétriques, toutes opérationnelles le 24 avril 1982.

Les relevés matériels intégrés au programme sur le terrain comprennent l'établissement des types et des dimensions de la structure le long du cours d'eau, la matérialisation de coupes transversales représentatives du canal de la rivière et de sa plaine inondable et l'établissement de renvois de ces coupes au système des coordonnées du Nouveau-Brunswick.

La régularisation du débit et des niveaux des crues au cours des débits de pointe est assurée le long de Walker Brook par quelque 32 ponts et ponceaux ainsi que par quatre déversoirs,

de la sortie d'eau de la rivière jusqu'à la limite en amont de notre zone d'étude. Un grand nombre de ces ouvrages constituent un resserrement considérable du cours d'eau et sont souvent engorgés, ce qui accroît les risques pour les résidents de la plaine inondable adjacente. Toutefois, un certain nombre de ces installations situées sur les tronçons en amont sont des traversées privées qui pourraient facilement être emportées par les eaux au moment des débits de pointe. On a obtenu, au cours des relevés sur le terrain, les dimensions matérielles et les élévations de contrôle de tous les ouvrages. On a relevé en tout 43 coupes transversales de la rivière Walker Brook, de son confluent avec la rivière Restigouche jusqu'à l'intersection des routes 170 et 175, en amont de Val d'Amour, près de McKendrick. On a arrêté la localisation des coupes transversales en fonction des changements opérés dans la pente, de la zone de la coupe ou des aspérités du canal. On a en outre procédé à la mesure d'autres coupes transversales près de ponts et de ponceaux.

Une comparaison établie entre les relevés obtenus en 1978 et en 1982 révèle que des changements sont survenus dans les coupes transversales de certaines zones en raison de l'érosion ou du dépôt de sédiments dans le cours d'eau. Par exemple, environ de 1 à 1,5 mètre de sable et de gravier s'est accumulé le long du tronçon du canal en aval de la rue Dover. Cette constatation reflète jusqu'à un certain point les observations faites par les résidents de la localité, à savoir qu'une accumulation importante de sédiments a fortement restreint la capacité hydraulique du pont de la promenade Gallant. Il semble qu'une accumulation substantielle de sable et de gravier à cet endroit a fait monter le lit du canal.

L'analyse des données de base et des relevés indique que deux tronçons principaux caractérisent le canal de Walker Brook: l'amont et l'aval de la route 11 près du parc provincial Sugarloaf. En amont du parc provincial, le canal a davantage

conservé son état naturel et après la route 270, la rivière coule dans une pente raide en serpentant à travers Val d'Amour. Les rives du canal sont relativement propres et bien définies tandis que son lit est fait de pierres et de cailloux. Le canal ne contient que peu de végétation en raison de la rapidité du débit et de la pente abrupte du lit du cours d'eau, sauf en certains endroits en amont de Val d'Amour et près de la traversée du passage des motoneiges de Sugarloaf.

En aval de Sugarloaf, les effets de la main de l'homme sur le canal de Walker Brook sont plus divers et on dénombre de grandes zones protégées par des perrés. D'autres zones sont touchées par l'érosion des berges et par le dépôt de sédiments dans la partie inférieure du cours d'eau.

Également, les caractéristiques de la plaine inondable de Walker Brook diffèrent en amont et en aval du secteur de Sugarloaf et de la route 11. En amont de Sugarloaf, les zones qui surplombent les rives sont fortement recouvertes d'arbres et de buissons. La plaine inondable est relativement étroite et une vallée abrupte limite le cours d'eau. En général, la partie supérieure de la rive possède une plus grande capacité d'écoulement puisque le canal longe la route 170. Cependant, sur la partie supérieure de la rive ouest on retrouve des boisés et des habitations qui tendent à restreindre l'écoulement de la rivière.

2.3 Hydrologie

La surface de la zone submergée par les crues est reliée à l'importance du débit de pointe, qui, à son tour, est fonction des conditions climatologiques qui prévalent dans le bassin hydrographique. Il faut effectuer beaucoup de recherches pour estimer d'une manière raisonnable les débits de pointe en se fondant sur les meilleures techniques disponibles. Aux fins de la présente étude, nous avons élaboré et comparé deux méthodes d'évaluation:

- i) l'évaluation statistique des débits de pointe en nous fondant sur les enregistrements des débits du cours d'eau;
- ii) la simulation par ordinateur des débits de pointe en nous inspirant des enregistrements climatologiques.

Nous avons eu recours à ces deux techniques à tour de rôle afin de disposer d'évaluations fiables des débits de pointe qu'on retrouve tous les vingt ans (1:20) et tous les cent ans (1:100).

i) Les évaluations statistiques

On détermine normalement les inondations à cartographier au moyen d'analyses de fréquence en s'appuyant sur l'hypothèse que les inondations constituent des manifestations naturelles qui peuvent se matérialiser par une courbe de distribution de probabilité. Pour réaliser cette analyse de fréquence, il faut obtenir et analyser tous les renseignements consignés, vérifier la possibilité que se forment des embâcles de glaces et de bois ainsi que recourir à une technique pertinente de distribution théorique des probabilités.

L'analyse de la fréquence des crues se produisant à un seul endroit pour un cours d'eau soumis à une étude n'est valable que si les données enregistrées couvrent une longue période et sont fiables. Il a été impossible de satisfaire à ces critères puisque les données fournies par la station hydrométrique de Walker Brook ne remontent pas assez loin dans le passé pour permettre une analyse statistique sûre. Par conséquent, on s'est fondé sur une analyse régionale de la fréquence des crues (Acres 1977), étude qui regroupe plusieurs analyses d'emplacements uniques en s'appuyant sur des données provenant des stations hydrométriques de la région.

Afin de cerner l'ampleur des inondations, on a aussi utilisé la méthode du transfert des évaluations des débits de pointe

obtenus pour un autre cours d'eau situé près de notre zone d'étude et qui possédait suffisamment de données enregistrées et fiables. Nos recherches ont révélé qu'il est possible d'appliquer au bassin hydrographique de Walker Brook les évaluations des débits de pointe afin de confirmer ces dernières avec plus de certitude.

ii) Les simulations par ordinateur

Dans notre effort d'évaluation des débits de pointe, nous avons aussi utilisé les enregistrements climatologiques, y compris ceux touchant les orages et les fontes des neiges historiques de la région, combinées aux précipitations. On a calculé le ruissellement découlant de ces conditions ainsi que les débits du cours d'eau au moyen d'un modèle hydrologique approprié du bassin hydrographique. Dans ce cas, la technique informatique appelée HYMO a servi à reproduire les débits de pointe. Ce modèle retient tous les facteurs qui ont une incidence déterminante sur les caractéristiques du ruissellement et a été vérifié en comparant les débits de pointe simulés et ceux enregistrés au cours des inondations historiques survenues le long de la rivière Walker Brook.

La composante hydrologique de l'étude comprenait aussi la vérification de la réaction du bassin hydrographique de Walker Brook aux grandes manifestations climatiques telles que les orages, la fonte rapide des neiges ou la combinaison de ces deux éléments. Outre la simulation par ordinateur de la période de récurrence de 1:20 et de 1:100, on a aussi évalué les débits de crues à l'air libre, les débits de pointe associés aux estimations des précipitations maximales, telles que les détermine le Service de l'environnement atmosphérique d'Environnement Canada, ainsi que les précipitations maximales associées à l'ouragan Belle qu'on a transposées au-dessus du bassin hydrographique.

La comparaison établie entre les évaluations statistiques et les résultats obtenus par le modèle informatique confirme que les débits de pointe prévus par l'analyse régionale de fréquence des inondations sont raisonnables et représentent des valeurs dignes de foi et dont on peut se servir pour le bassin de Walker Brook.

Le tableau suivant résume l'évaluation des débits de pointe obtenus à plusieurs endroits de la rivière Walker Brook.

<u>Endroit</u>	Débits de pointe (en mètres cubes par seconde)	
	tous les 20 ans	tous les 100 ans
(a) Walker Brook près de sa sortie	57.9	82.9
(b) Walker Brook près du mont Sugarloaf	53.4	76.5
(c) Walker Brook en amont de Sugarloaf	42.3	61.1

Si les précipitations maximales relatives à l'ouragan Belle s'étaient abattues sur le bassin hydrographique de Walker Brook, on a estimé que les débits de pointe qui en auraient résulté auraient été de l'ordre de 100 mètres cubes à la seconde. De plus, on a aussi indiqué que le débit de pointe qui résulterait des plus grandes précipitations qu'on puisse prévoir pourrait être de l'ordre de plus de 350 mètres cubes à la seconde. Il faut se rappeler qu'un débit de cette importance ne surviendrait que très rarement et entraînerait une inondation dévastatrice.

2.4 Hydraulique

Pour réaliser le calcul des profils de la surface des eaux dans le cas d'inondations se produisant tous les 20 ans ou tous les 100 ans, on a utilisé un programme d'ordinateur que l'on appelle modèle de la longueur du remous HEC-2. Ce modèle

tient compte d'un certain nombre des facteurs hydrauliques et décrit le canal et la plaine inondable du tronçon qui fait l'objet de notre étude. C'est un des modèles les plus utilisés et les mieux reconnus en ce qui concerne le calcul des profils des crues.

Les paramètres d'intrant ont varié le long du tronçon étudié en fonction des relevés obtenus sur le terrain, comme nous l'avons précisé à la section 2.2, et en fonction des résultats de l'analyse hydrologique, comme nous l'avons indiqué à la section 2.3. On peut à l'avenir recourir au modèle HEC-2 pour jauger les effets des changements futurs, y compris les projets d'amélioration des installations hydrauliques ou les projets de fosse longeant le tronçon en question.

Il a été malheureusement impossible de confronter les profils des crues obtenus par ordinateur avec ceux observés le long du tronçon puisque les conditions propices à une crue ne se sont pas produites pendant nos recherches sur le terrain.

Une analyse du débit et des conditions des crues propres à la hauteur des inondations se produisant tous les 20 ans et tous les 100 ans a révélé qu'au cours des débits de pointe, le canal déborderait d'une manière importante aux endroits suivants (figure 2):

- le vieux barrage en aval du parc Sugarloaf (les crues s'écouleraient sous le passage supérieur de la route 11 pour rejoindre partiellement le canal principal aux environs du garage municipal; le reste des crues passerait le long de la dérivation de la route de Val d'Amour avant de rejoindre le canal principal près de la rue Alexander);
- le passage supérieur du pont de la route 11 à la route 270 (les eaux s'écouleraient dans le fossé longeant la

route 11 avant de regagner le canal principal près de la promenade Gallant).

Les débordements du canal et de la plaine inondable qui se produisent à ces endroits tendent à réduire les débits de pointe et les niveaux des crues aux endroits situés immédiatement en aval. Toutefois, le niveau des crues remonte de nouveau à l'endroit où les eaux qui débordent regagnent le canal principal de Walker Brook. On a élaboré des courbes de débits jaugés pour chaque endroit où la rivière sort de son lit afin de déterminer l'importance du débordement et son effet sur les niveaux des crues en aval.

Les analyses hydrauliques portent aussi sur la formation possible d'embâcles causés par les glaces. On a découvert que le plus grand amoncellement de glaces se forme chaque année, semble-t-il, en aval de la jauge des Relevés hydrologiques du Canada, qui est située près du mont Sugarloaf. Il existe donc une plus grande possibilité de formation d'embâcles de glaces dans la partie inférieure de la zone étudiée. À la suite de ces recherches, on a découvert que des embâcles importants composés de glaces et de débris se forment principalement aux endroits où le canal comporte des obstacles faits de main d'homme.

On a aussi déterminé que le principal effet des embâcles causés par les glaces et les débris se fait sentir au chemin Village et à la promenade Gallant ainsi que dans la courbe du canal située immédiatement en amont de la rue Gérard. Il pourrait se produire à ces endroits une obstruction totale des ouvrages d'adduction des eaux, ce qui entraînerait un débordement au-delà de la route. Cependant, l'effet de remous en amont se limiterait passablement à ces deux endroits à cause du profil relativement en pente du canal.

2.5 Analyse des résultats

On a cerné les caractéristiques types du canal et de la plaine d'inondation des divers tronçons de la rivière Walker Brook. On peut dire en général que le canal compte deux tronçons, un qui se situe en amont de la route 11, près du parc provincial Sugarloaf, et qui va au-delà de Val d'Amour, et l'autre qui s'étend en aval de cette première zone et qui traverse la ville de Campbellton. Dans le secteur aval, on dénombre un grand nombre d'habitations et d'immeubles commerciaux implantés dans une plaine inondable relativement large. On croit que les risques de dommages à ces bâtiments sont très élevés. Dans le secteur amont, le canal est étroit et assez profond et les risques que représentent les inondations se limitent essentiellement à un certain nombre de maisons et de petits ponts jetés par-dessus le canal, surtout dans le secteur de Val d'Amour.

Du confluent de Walker Brook et de la rivière Restigouche jusqu'au chemin Village, on a noté que les risques d'inondation sont très faibles puisque les hauteurs des crues susceptibles de se produire tous les vingt ans ou tous les cent ans ne touchent pas les zones d'aménagement de l'agglomération.

En amont du chemin Village jusqu'à la rue Dover, les risques d'inondation sont plus grands puisque l'écoulement au-delà du chemin Village et de la promenade Gallant se produit tant pour la période de récurrence de 20 ans que pour celle de 100 ans. Il est aussi possible que des embâcles causés par les glaces et les débris se forment et entraînent le refoulement des crues par-dessus la route si les ouvrages d'adduction des eaux du chemin Village et de la promenade Gallant sont entièrement obstrués. Toutefois, peu de bâtiments sont touchés directement par les crues à ce tronçon.

En ce qui concerne le tronçon s'étendant du chemin Dover jusqu'à la rue Pine, il présente des risques moyens d'inondation.

Étant donné que les eaux se déversent du côté sud de la route 11, l'écoulement dans la rivière Brook au nord de la route (écoulement pondéré) se limiterait au canal et à la plaine inondable tandis que les crues (niveau non pondéré) qui se produisent une fois en cent ans (aucun débordement dans le fossé de la route) submergeraient certains ouvrages érigés dans le voisinage des rues Pine et Ila. En amont de la rue Pine jusqu'à la zone comprise entre la route 11 et le prolongement de Val d'Amour (la limite de la zone cartographiée), les risques d'inondation sont très élevés. L'étendue de la zone de crues susceptibles de se produire une fois en cent ans englobe environ 160 ouvrages. Par contre, si le fossé qui longe la route 11 servait de déversoir, le risque d'inondation s'en trouverait très réduit et les crues ne toucheraient qu'une quarantaine d'ouvrages le long de ce tronçon.

En amont de la route 11, les débits de pointe se confinerait normalement au canal naturel et à la plaine d'inondation, à l'exception des zones de débordement mentionnées ci-devant. Les principaux risques d'inondation relevés dans les tronçons en amont semblent être le fait de la grande vitesse de déversement et de la possibilité que soient emportés par les crues les quelques ponts de bois qui traversent le cours à divers endroits de la zone étudiée.

3. LES MESURES CORRECTIVES POSSIBLES

Une fois connues les diverses zones susceptibles d'être inondées le long de la rivière Walker Brook, comme l'indique la partie précédente de ce rapport, il est maintenant possible d'envisager certaines mesures qui atténueraient les problèmes causés par les inondations.

On peut classer en général les composantes d'un plan de réduction des dommages causés par les inondations en:

- mesures autres que les ouvrages de protection et qui visent à réduire les pertes éventuelles en décourageant

l'aménagement des terres sujettes aux inondations et en encourageant à protéger les bâtiments existants;

- ouvrages qui influent sur les causes qui contribuent aux problèmes qu'entraînent les inondations.

La présente étude n'avait pas pour objet de présenter une analyse détaillée des mesures correctives à prendre; ce sera l'objectif des second et troisième volets du projet d'étude de Walker Brook. Toutefois, en se fondant sur les résultats obtenus dans la présente étude, on propose d'analyser dans une étude ultérieure certaines mesures propres à régulariser les crues, mesures exposées dans les sections qui suivent.

3.1 Les mesures autres que les ouvrages

Afin de réduire la possibilité d'accroissement des dommages occasionnés par les crues, on pourrait élaborer et imposer des règlements régissant la plaine inondable afin de limiter l'aménagement dans les zones qui présentent des risques d'inondation le long de la rivière Walker Brook. Ces règlements pourraient s'inspirer de la notion de la double zone: une zone présentant des risques élevés d'inondation et une zone périphérique. Dans la première zone, il serait interdit d'y ériger des bâtiments tandis que dans la seconde, on pourrait autoriser l'aménagement de nouveaux terrains en fonction du degré de risques qu'ils présenteraient et de la mise en oeuvre de méthodes approuvées de protection des nouveaux bâtiments contre les crues.

Parmi les autres mesures visant à réduire les demandes d'indemnisation des dommages occasionnés par les crues, on note la réinstallation des entreprises situées dans la zone qui présente de très grands risques ou la mise en place d'un programme de subventions destinées à aider les propriétaires de bâtiments à protéger leurs terrains et leurs bâtiments. Ces méthodes, ou toutes autres mesures, devraient tenir compte de la rentabilité, des avantages et de leur incidence sociale ou écologique.

3.2 Les ouvrages

On devrait envisager, à titre de mesures de protection, de construire les ouvrages suivants le long de Walker Brook par ordre de priorité:

1. La dérivation du débit à la route 11. La dérivation actuelle du cours d'eau le long de la route 11 pourrait faire l'objet d'une analyse plus poussée et qui permettrait de vérifier la faisabilité d'un ouvrage de dérivation permanent.
2. L'enlèvement des obstacles à l'écoulement des eaux. Cette solution peut signifier le retrait des obstacles suivants à l'écoulement:
 - (a) le barrage désuet à démolir près du mont Sugarloaf;
 - (b) les ponceaux du chemin Village à remplacer (si une analyse coûts/avantages en révèle la viabilité);
 - (c) le pont de la promenade Gallant à remplacer, et
 - (d) le barrage désuet à démolir en aval du chemin Village.
3. La construction de digues ou de berms le long de tronçons choisis du canal. On peut aussi envisager d'ériger une berme de protection contre les crues des deux côtés du canal immédiatement en aval de la route 11. Cet ouvrage protégerait les zones inondables situées entre la route 11 et la rue Dover et signifierait l'installation d'un ponceau double ou l'agrandissement du ponceau actuel de la rue Dover.
4. Des travaux d'aménagement du canal qui pourraient comprendre:
 - (a) l'élimination de la végétation du canal en amont du chemin Village;

- (b) le dragage des pierres et des cailloux du canal en amont de la promenade Gallant et en aval de cette dernière jusqu'au chemin Village;
- (c) le redressement des parties du canal en aval de la rue Dover jusqu'à la promenade Gallant, surtout dans le secteur de la rue Gerrard; et
- (d) l'élargissement du canal (par exemple, dans la zone s'étendant en aval de la route 11 jusqu'en aval de la rue Dover).

Il semble qu'actuellement la construction de l'ouvrage de protection qui serait le plus souhaitable serait la dérivation d'une partie du débit le long de la route 11 ainsi que le remplacement des ponceaux du chemin Village et du pont de la promenade Gallant. Ces dernières mesures réduiraient la possibilité que se forment des embâcles de glaces et de débris à ces endroits.

La promenade Gallant constitue la principale voie d'accès aux hôpitaux provinciaux situés du côté de la route 11 qui donne sur Campbellton. On peut aussi envisager d'établir un nouveau tracé de route entre la zone d'inondation et la route 11. La possibilité de remplacer le pont de la promenade Gallant devrait donc également tenir compte du projet de nouveau tracé de la route qui éviterait la partie de la promenade Gallant sujette aux inondations.

Si la dérivation du cours d'eau le long de la route 11 s'avère faisable, il ne faudrait exécuter que des aménagements de peu d'importance aux ponceaux des rues Pine et Ila, Alexander et Dover.

4. CONCLUSIONS ET RECOMMANDATIONS

On a considéré depuis longtemps la plaine inondable de la rivière Walker Brook comme la zone du Nouveau-Brunswick la plus sujette aux inondations. L'aménagement continue de ce

secteur a aggravé les risques d'inondation. Ces derniers sont plus élevés le long du tronçon entre le parc provincial de Sugarloaf et le chemin Village et surtout dans le secteur entre les rues Dover et Pine. De plus, il existe un certain nombre de ponts de taille insuffisante dans la zone de Val d'Amour qui peuvent être emportés et qui l'ont été au cours d'inondations.

Les inondations qui causent le plus de dégâts surviennent généralement au cours du ruissellement du printemps. Cependant, au printemps de 1982, période où l'on a réalisé les relevés sur le terrain, il ne s'est produit aucun écoulement important dans le cours d'eau ni de crues dans la zone en question.

Afin d'étalonner correctement le modèle hydrologique et le modèle de la longueur de remous, il importe d'utiliser des crues qui entraînent un écoulement semblable à celui qui se produit tous les vingt ans ou tous les cent ans. Nous proposons donc de maintenir le plus longtemps possible la collecte et l'enregistrement de données relatives à la météorologie, aux niveaux d'eau, au débit, à la formation d'embâcles causés par les glaces ainsi qu'à l'épaisseur de la glace de manière à pouvoir vérifier les calculs futurs en ce qui concerne les pires conditions d'inondation.

Il est nécessaire d'établir une cartographie détaillée afin de mesurer avec précision la nature et la portée des crues nominales dans la plaine inondable. Les cartes disponibles pour notre étude et dont l'échelle est de 1:1200 ne couvrent que la région qui s'étend légèrement en amont de la rue Dover près des limites de la ville de Campbellton. Nous proposons donc qu'on obtienne, aux fins énoncées ci-devant, des cartes topographiques à jour pour les terres sujettes aux inondations le long de la rivière Walker Brook. Ces cartes constitueront également le point de départ de l'élaboration éventuelle de

cartes des risques d'inondation. En outre, il faudrait procéder à une étude des schémas d'écoulement des eaux de surface afin de cerner avec précision la portée des risques d'inondation et des mesures correctives possibles.

Une fois que tous les organismes engagés dans le projet de Walker Brook auront passé en revue les résultats obtenus à la suite du premier volet de l'étude, nous proposons d'entreprendre le plus tôt possible le volet suivant de l'étude. Il s'agirait alors de procéder à un inventaire sur le terrain destiné à repérer et à classer les ouvrages sujets aux crues et à établir les coûts et les avantages d'un programme de régularisation des eaux. Entre-temps, les autorités qui ont le pouvoir de déterminer l'aménagement le long de Walker Brook devraient être informées des limites des zones sujettes aux inondations et déterminées par l'étude de manière à empêcher la construction de nouveaux bâtiments qui pourraient être endommagés par les inondations.

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Canada - New Brunswick
Flood Damage Reduction Program

Hydrotechnical Study Of The Walker Brook Flood Plain

Restigouche County

Main Report

January , 1983



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LIST OF SYMBOLS

A	Drainage area (km^2)
AMC	Antecedent Moisture Condition
As	Area swamps and lakes (km^2)
B	Surveyed bridge section
C.K.	Coefficient of kurtosis
C.S.	Coefficient of skew
CL	Confidence limit
CMP	Corrugated metal pipe
cms	Cubic metres per second
CN	Average Soil Cover Complex Number
D	Surveyed dam or weir section
d/s	Downstream
DA	Drainage area (km^2)
f(tide)	flow capacity is a function of tidal level
FDTs	Proportion of annual maximum floods occurring in March, April, May period at drainage basin centroid
g	Acceleration due to gravity (m/s^2)
h	Depth of flow (m)
K	Hydrograph recession parameter (hrs)
L	Watershed length
LN	Log Normal probability distribution
LP3	Log Pearson Type III probability distribution
M	Snowmelt (mm)
M_{cc}	Melt due to condensation and convection (mm)
M_g	Melt due to heat transfer from the ground (mm)
M_p	Melt due to rainfall (mm)

M_{rl}	Melt due to long wave radiation (mm)
M_{rs}	Melt due to short wave radiation (mm)
n	Manning's roughness coefficient
N	Number of years
P	Precipitation amount (mm)
PMP	Probable Maximum Precipitation (mm)
Q	Amount of runoff (mm)
Q_p	Annual maximum instantaneous peak flow (m^3/s)
$\overline{Q_p}$	Mean annual maximum instantaneous peak flow (m^3/s)
Q'_T	Flood peak of return period T (years)
QC_T	Design flood peak at a specific site for the T -year return period calculated using the appropriate regional relationship (m^3/s)
QC'_T	Flood peak at hydrometric station for the T -year return period calculated using the appropriate regional relationship (m^3/s)
Q_P	Annual flood peak (m^3/s)
QP_{100}	Maximum instantaneous 100 year return period flow rate (m^3/s)
QP_{20}	Maximum instantaneous 20 year return period flow rate (m^3/s)
QP_n	Cumulative moving mean at year n
QT	Derived design flood peak at selected locations in the study area for return period T -years (m^3/s)
R	Hydraulic radius
S	Field Surveyed section
s	$\frac{1000}{CN} - 10$
$S.D.$	Standard deviation
S_f	Boundary frictional effect
SLP	Slope of watershed
S_o	Bottom channel slope
SQP	Standard deviation of maximum instantaneous peak flow series

T_p	Hydrograph time to peak parameter (hrs)
3PLN	3 Parameter Log Normal probability distribution
U.C.L.	Upper 95% confidence limits
u/s	Upstream
V	Velocity in direction of flow (m/s)
W	$\frac{A-A_s}{A}$
w	Watershed width
X	Distance in direction of flow (m)
	Summation

Acknowledgement

The information and conclusions presented in this report were derived with assistance from several individuals and organizations.

The following members of the Walker Brook Project Committee provided significant input and direction throughout the study:

- | | |
|----------------------|--|
| - Mr. J. G. Lockhart | Environment New Brunswick |
| - Mr. J. D. Keefe | Inland Waters Directorate,
Environment Canada |
| - Ms. E. Langley | Inland Waters Directorate,
Environment Canada |
| - Mr. J. E. Peters | (retired) Inland Waters
Directorate, Environment Canada |
| - Dr. N. D. Elhadi | Environment New Brunswick |
| - Mr. B. C. Burrell | Environment New Brunswick |
| - Mr. D. Sullivan | New Brunswick Department of
Transportation |

The background information presented in this report was obtained from several sources including personnel from the Atmospheric Environment Services; the Water Survey of Canada; the Water Planning and Management Branch and the Tides and Water Levels Branch - all of Environment Canada. Background data collected by the Water Resources Branch of Environment New Brunswick also proved most useful during these investigations.

The discussions and input to the study by Mr. Leo Berthelot, Manager, Water and Sewerage Department for the City of Campbellton are also gratefully acknowledged.

We would also like to express our appreciation for the time and effort of all others who contributed to this project by way of information, discussions and otherwise.

1.0 INTRODUCTION

1.1 General

Historically, the development of many urban centres in Canada and New Brunswick has taken place on flood prone lands. These lands were developed by the first settlers because of their agricultural productiveness and, in some cases, so the the river could be utilized as the main transportation route. These early uses of the floodplain have evolved into present day highly urbanized communities which still attempt to utilize floodplain lands. An increasing trend towards urban developments in Canada has resulted in an increased potential for higher flood losses. A nation-wide survey of potential flood hazards (Burton, 1965) has indicated that more than 200 communities in Canada have some developments located in flood hazard areas. In particular, the serious consequences of flooding in New Brunswick and more specifically, in the Campbellton area of the Walker Brook floodplain have been well documented in a number of reports (Environment Canada 1976; Folster 1980).

There is a continuing pressure to develop additional lands in the Walker Brook floodplain areas. As a result, land and property values in the flood hazard areas continue to escalate as existing structures are replaced or remodelled. This has led to an increased potential for future flood losses in the Walker Brook floodplain (Table 1.1). The continued development of flood prone lands has also led to a corresponding demand for an increased level of protection from flood damages. In some areas this has resulted in pressure for increased flood damage compensation or modifications to flood protection systems. However, protective works are very costly to construct and do not provide absolute protection from flood damages. In addition, structural measures to provide flood protection tend to invite additional development in the floodplain, and hence, an alternative approach to control development in the flood prone areas is more desirable.

In recent years, the Province of New Brunswick and the Federal Government have made compensation payments to flood victims totalling several million dollars. Total flood losses across Canada are estimated to be approximately two hundred million dollars in the 1970's.

On March 31, 1976, the Province of New Brunswick and the Government of Canada entered into a General Agreement Respecting Flood Damage Reduction. The objectives of this Agreement are to reduce the potential for flood damages in floodplains and along the shores of lakes, rivers and the sea. This Agreement also recognizes that the potential for future flood damages can be reduced by controlling the areas prone to flooding.

The General Agreement Respecting Flood Damage Reduction allows the two levels of government to enter into a number of sub-agreements on specific aspects of flood damage reduction, including but not limited to, land use planning, flood proofing, flood risk mapping, flood forecasting, flood control works and flood studies.

To provide for the identification and delineation of flood prone areas in New Brunswick, the "Agreement Respecting Flood Risk Mapping" was also signed on March 31, 1976 and amended in July 16, 1981. Under the terms of this sub-agreement, a number of flood prone areas in the Province are to be mapped and flood risk zones delineated and ultimately designated as areas where the Federal and Provincial governments have agreed to restrict their funding of new development.

In order to reduce the amount of flood damage to moveable property, "An Agreement Respecting Flood Forecasting" was made as of August 30, 1977. The objective of this sub-agreement is to develop a flood forecasting capability for the New Brunswick portion of the Saint John River Basin. Since 1977, the Saint John River Forecast Centre has used modern facilities and techniques to provide water level and river flow forecasts for several locations in the Saint John River Basin.

The Government of Canada, the Province of New Brunswick, and the City of Saint John entered into an agreement for implementation of various measures aimed at reducing the flooding problem along Marsh Creek. Structural flood damage reduction measures included the building of a flood control reservoir, channel cleaning and the replacement of undersized structures. These structural measures were supported by non-structural measures such as a watershed management plan and a City of St. John by-law controlling development on the Marsh Creek floodplain.

Due to the serious flooding that occurred at Saint John by Marsh Creek, at Fredericton by the Saint John River and at Campbellton by Walker Brook and the Restigouche River, "An Agreement Respecting Studies for Flood Damage Reduction" was made as of March 31, 1976 and amended on July 16, 1981. This sub-agreement allows for studies to be carried out in order to determine what the best measures to reduce flood damages in a particular area would be. These studies may include background hydrotechnical studies. Hydrotechnical studies, such as this one, are necessary in order to allow the areal extent and seriousness of flooding to be evaluated preceding the planning and undertaking of flood damage reduction measures.

A multi-phased project for reducing potential flood losses in the Walker Brook floodplain has been proposed. Each phase is described in the Terms of Reference (Canada-New Brunswick, 1981). The following phases for reducing flood damages in the Walker Brook floodplain are presently envisaged.

1. Phase 1 - Hydrotechnical

- Climatology
- Hydrology
- Hydraulics

2. Phase 2 - Planning

- Interim designation
- Stage damage relationships
- Identification of alternatives
- Identification of feasible alternatives

3. Phase 3 - Evaluation

- Evaluation of flood damage reduction measures
- Preliminary design
- Environmental impact analysis
- Preliminary cost estimates
- Benefit-cost analysis
- Flood risk mapping
- Public meetings

4. Phase 4 - Design

- Design of flood damage reduction measures
- Detailed cost estimates

5. Phase 5 - Implementation

- Construction

The primary purpose of the present study was to undertake Phase 1 of the anticipated investigations. This includes the determination of flood discharge and water levels and the identification of flood prone areas along Walker Brook. The need for each of the subsequent phases will depend on the findings of the immediately preceding phase of the overall investigations.

1.2 Authorization and Scope of Study

The agreements previously mentioned provide for the establishment of two committees, the Steering Committee which is responsible for general administration of the agreements and the Project Committee which provides technical support to the Steering Committee. On March 9, 1982, Spear Macdonald & Associates Limited, in association with Cumming-Cockburn & Associates Limited were commissioned by the New Brunswick Department of the Environment on behalf of the Steering Committee to undertake Phase 1 - Hydrotechnical Study of the Walker Brook Floodplain. As described in the Terms of Reference, the main objective of this investigation was to develop the 1:20 and 1:100 year return period flood hydrographs and backwater profiles for the study area.

The following points summarize the overall scope of the investigations undertaken for Phase 1.

1. Description of the climatology and hydrology of the Walker Brook watershed
2. Analysis of the hydrologic response of the Walker Brook watershed to storm events.
3. Design, coordinate and manage a field program for the purpose of collecting hydrologic and hydraulic data for model calibration and validation

4. Determine 1:20 and 1:100 year recurrence interval open water flood hydrographs and backwater profiles from the confluence up to the community of Val D'Amour.
5. Plot the 1:20 and 1:100 year return period flood lines on existing topographical maps to determine their areal extent
6. Assess the implications of spill from the main channel
7. Undertake sensitivity analyses of peak flow estimates and backwater profiles
8. Assess the significance of ice jamming, debris jamming and other hydraulic factors affecting flood lines
9. Identify possible remedial measures for flood management and flood damage reduction to be analysed in detail in future phases of the flood hazard investigations.

The scope of the study is described in detail in the Terms of Reference (Canada-New Brunswick 1981).

1.3 Study Area Description

Walker Brook is a relatively small stream (drainage area 67.4 km² to its outlet) flowing through the community of Val D'Amour, part of the Village of Atholville, and the City of Campbellton. The stream originates in the highlands south of Val D'Amour and flows northerly along a steep channel to discharge into the Restigouche River at Campbellton. The general location of the study area is depicted on Figure 1.1.

For the purposes of the hydrologic and climatologic investigations, the study area encompasses the entire Walker Brook watershed in order to allow the hydrologic response of the watershed to specific storm events to be determined. Hydraulic investigations were undertaken along the floodplain and channel of Walker Brook from its confluence with the Restigouche River upstream to a point just south of the community of Val D'Amour. The watershed and study area are shown on Figure 1.2.

Within the study area two reaches along Walker Brook have been identified as having a high flood potential (Canada-New Brunswick 1981). One major floodplain is located between the Sugarloaf Provincial Park and the confluence of Walker Brook with the Restigouche River. The other area is the stream reach extending from Sugarloaf upstream through the community of Val D'Amour.

The floodplain along Walker Brook has long been considered as one of the most flood prone areas in the Province of New Brunswick and as such, was high on the priority list in March, 1976 when the General Agreement Respecting Flood Damage Reduction and the Flood Risk Mapping and Studies Agreements were signed. There has been extensive development of the Walker Brook floodplain in recent years in the form of single family homes, apartments, low rental units and mobile homes. Additionally, a large shopping mall, municipal offices and garage and several business establishments have been located near the stream and Provincial tourist parks facilities have been constructed.

The trend toward greater development in the future is also apparent, and several large tracts of land have been staked out for subdivision. Reclamation of interval land has also been accomplished by land filling in flood prone areas. With this almost continuous development, a number of access structures have been installed to permit travel across the Brook and some of these are hydraulically under-designed, creating impediments to flow and increased backwater flooding.

There has also been considerable development in the Val D'Amour area. This is largely residential with homes being built in close proximity to the stream. In many cases, these locations appear to be prone to flood damage under high water conditions. Numerous small bridges have been built across Walker Brook in the Val D'Amour area and these are susceptible to flood damages.

Other areas of the watershed are largely undeveloped and are comprised of steeply sloping topography with extensive forest cover. Historical flood problems have been confined mainly to developed areas along the main channel.

1.4 Overview of Study Methodology

As indicated previously, the basic purpose of this investigation was to provide the 1:20 and 1:100 year open water flood profiles and floodplain extent for Walker Brook from its confluence with the Restigouche River to Val D'Amour. The accurate determination of flood profiles along the study reach depends on several hydrological and hydraulic factors, including the following:

- historical flood conditions in the study area
- climatological characteristics of the Walker Brook watershed; including rainfall and snowmelt characteristics
- existing and future land use in the watershed including degree of urbanization, etc.
- peak discharge rates associated with the 1:20 and 1:100 year return period floods
- effects of tides, ice jams, debris jams and other hydraulic factors along the study reach
- existing stream channel and floodplain hydraulic characteristics and man-made changes such as bridge and channel constrictions, etc.
- natural and artificial flood storage in the study area
- the effects of natural and artificial spill from the channel.

The complex interrelationships between the above mentioned factors have been considered in the course of undertaking our climatologic, hydrologic and hydraulic investigations.

The first step in the investigation was the collection and review of available background information and existing data on climatologic and flood characteristics. These are discussed in Chapter 2.0, entitled "Background Information".

The next step was the determination of appropriate 1:20 and 1:100 year peak discharge rates. Alternative estimates by statistical and deterministic analyses were derived and compared as discussed in Chapter 3.0, "Hydrologic Analyses". Where appropriate, this included model calibration and sensitivity and error analyses in order to achieve an appropriate level of accuracy for the discharge estimates.

Thirdly, the peak flow estimates were converted to flood levels (profiles) along the study reach by means of a computer model of hydraulic characteristics. This is discussed in Chapter 4.0, including sensitivity testing of the most important hydraulic parameters.

Finally, the approximate extent of the flood hazard area was plotted on copies of existing topographic mapping for the lower part of the study area. It was then possible to identify potential remedial measures which might be considered in more detail in the future for alleviating the potential for future flood losses. These measures are identified in Chapter 5.

As required by the Terms of Reference, (Canada-New Brunswick, 1981), the "Hydrologic and Hydraulic Procedures for floodplain delineation developed by Environment Canada (Environment Canada, 1976) were used as basic guidelines throughout the course of these investigations.

TABLE 1.1

Summary of Recent Flood Problems Along Walker Brook*

<u>Stream Reach</u>	<u>Documented Flooding and/or Flood Damages</u>
A. Val D'Amour to Campbellton City	1947 - bridge almost destroyed on Val D'Amour Highway - Campbellton water reservoir overflowed 1973 - erosion on Val D'Amour Road - bridges washed out, Val D'Amour & Malauze Roads + others 1979-80 - private stream crossings washed out 1973-4, 1979 - remains of old dam north of Malauze Rd. caused backwater & flooding of road 1974 - culvert under Rte 270 west of Sugarloaf Park damaged - water over the road at the Rte 11 underpass (0.30 m deep) - walkway in Val D'Amour washed out
B. Rte 11 to Alexander St. (S9-S7)	1972, 1974 - water damage to several residences - Val D'Amour Rd. area residents have flood damage during every recorded flood event 1973 - St. Albert St. area flooded - two small bridges washed out - several houses damaged 1974 - \$5,500 claimed by local residents - St. Albert Street culvert washed out 1979 - \$12,000 damage to Val D'Amour Rd. residents - local property erosion
C. Alexander to Gerrard St. (S7-S4)	1974 - 1 m water over Val D'Amour Rd. & Pine St. intersection 1947 - old race track site under water (now Harvey Meadows subdivision) 1954 - race track road under water 1970 - culvert washed out in St. Albert area - minor property damage 1976 - huge "lake" found on Water St. to a depth of 0.3 m
D. Gerrard to Village Rd. (S4-S2)	1979 - 1 m of water over Gallant Dr. (125 m width of flow) 1949 - 0.3 m of water over Gallant Drive - debris & sedimentation at Gallant Dr. reduces culvert capacity 1973 - Gallant Dr. bridge abutments eroded and road closed - ten families evacuated
E. Village Rd. to Confluence (S2-outlet)	- .15 to .20 m of spill noted over the road during most floods - ice jamming at remains of old dam

* Source - Folster, 1981
 - Environment Canada, 1976



FIGURE 1.1

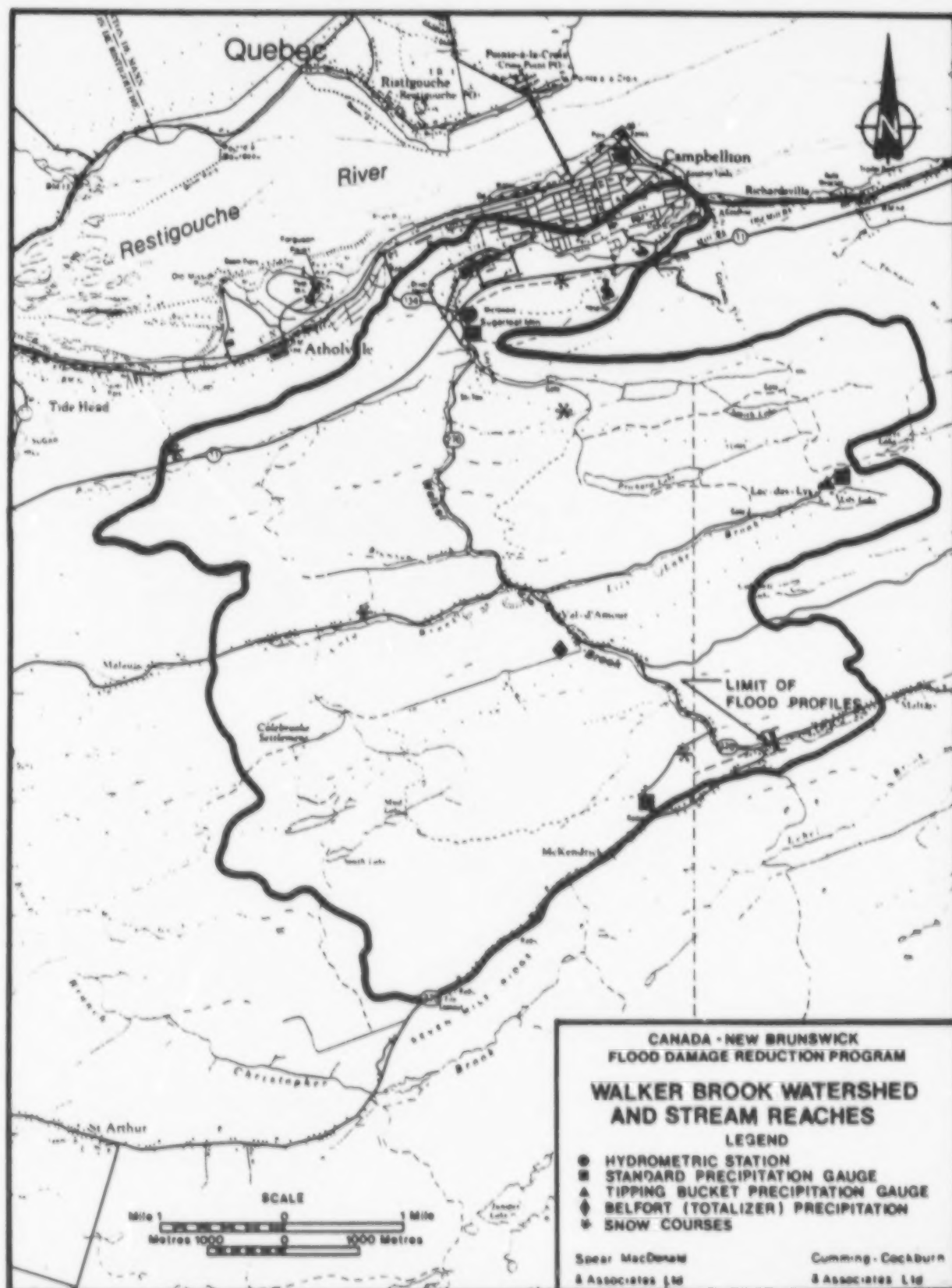


FIGURE 1.2

2.0 BACKGROUND INFORMATION

2.1 General

A few background studies documenting flood conditions in the Walker Brook watershed have been completed in recent years. One of the first projects undertaken under the Canada-New Brunswick Flood Damage Reduction Program resulted in a report entitled, "Regional Flood Frequency Analysis" (Acres, 1977). This investigation includes procedures for estimating the 1:20 and 1:100 year peak flows at various locations in the study area. These peak flow estimates are discussed in more detail in Section 3.2.2.

More recently a report entitled, "Climatological Data for the Walker Brook Basin in Relation to Flood Studies" was completed by the Atmospheric Environment Service (AES, 1981). This includes some analysis of peak rainfall and snowmelt conditions and is discussed in more detail in Section 3.3. It is noted that the Campbellton area experiences one of the highest snowfall accumulations in Canada and that floods could result from both spring snowmelt and summer rainstorm conditions.

The existing studies on the hydrology and climatology of the Walker Brook watershed have tended to confirm that a high potential exists for future flood losses in the study area.

A report on historic flooding in the Walker Brook study area was recently prepared for Environment New Brunswick (Folster, 1980). This confirmed the findings of a previous report on flooding events in New Brunswick which was undertaken by the Federal Government (Environment Canada, 1976). The general findings of these investigations concerning historical flood problems in the Walker Brook watershed are summarized briefly in the following paragraphs, and highlighted in Table 1.1.

2.2 Summary of Historical Flood Conditions

The existing information on historical floods on Walker Brook covers the period 1928 to 1982. The earliest records of flooding along Walker Brook suggest that flooding of structures usually occurred in the vicinity of Alexander Street and Val D'Amour Road and this is an area which continues to experience problems during flood events. Historically, most damage producing floods have taken place during the occurrence of the spring runoff. The most recent flood conditions occurred in the spring of 1979.

The review of background information reveals that the development of the Walker Brook floodplain has occurred mainly during the past ten years resulting in an increased amount of property damages. Recent records of historical flooding suggest that major floods or significant flood damage have occurred over the last 10-20 years, with an increased frequency of damage claims over the last ten years (Folster, 1980). Furthermore, long-time residents of the area believe that flooding has become more significant in recent years, corresponding to the continued development of the floodplain.

The flood of April 28-30, 1973 was reported as being the worst experienced in the area in the past half-century. This flood was caused by heavy rains combined with snowmelt, with rainfall amounts in the order of 100-125 mm occurring within a 2-day period. Table 1.1 presents a summary of flood susceptible areas along Walker Brook, together with an indication of the extent of damage and the cause of flooding.

In addition to economic losses due to flood damage, five children have drowned during flood conditions on Walker Brook over the past 50 years.

2.3 Existing Data

Field surveys of channel and floodplain characteristics were carried out in the study area by Environment New Brunswick (King, 1978). This included the collection of channel and floodplain cross-sections, determination of the stream channel profile, and measurements of bridges, culverts and hydraulic structures. However, due to sedimentation and erosion processes and the resulting

changes in cross-sectional properties since the time of the survey, it was necessary to update this information as part of the present investigations as described in Section 4.3.1.

For example, a comparison of cross-section measurements from these surveys (1978 and 1982) has indicated that about 1 to 1.5 m of sand and gravel has accumulated along the channel reach downstream of Dover Road (B7). This is consistent to some extent with the observation of local residents that significant accumulation of sediment has severely restricted the hydraulic capacity of the Gallant Street Bridge. Apparently a significant amount of sand and gravel has accumulated to raise the bed of the channel at this location.

Some data describing local hydrologic and climatologic characteristics were also available from several agencies. Background measurements on streamflow, water levels, tidal levels, rainfall, snowfall, snow accumulation and ice cover characteristics were also collected and reviewed as discussed briefly in the following paragraphs.

Streamflow records within the study area are measured by the Water Survey of Canada for the hydrometric station located on Walker Brook near Sugarloaf Provincial Park (Gauge No. 01BJ009). This data is available for the period November, 1980 up to date, and is published as mean daily discharge. A summary characterizing available discharge measurements is presented in Table 2.1. Additional discharge measurements are also available for nearby streams. For example, discharge measurements have been taken by the Water Survey of Canada on the Eel River near Eel River Crossing since June, 1967 (Gauge No. 01BJ004). Mean discharge rates for this drainage area (88.6 km²) are also summarized in Table 2.1. The mean annual discharge on the Eel River and Walker Brook is observed to be about 2.1 m³/s and 1.7 m³/s respectively.

Water level records and rating tables are also available at the gauge locations. In addition, some previous measurements on peak water levels were collected using a series of crest gauges located along the lower portion of Walker Brook. However, this data was limited due to vandalism of the gauges and non-occurrence of peak flows during the recording period and was not used in this study.

Some information on the influence of tides on water levels is also available near the confluence of Walker Brook with the Restigouche River. Tidal information is published under the authority of the Canadian Hydrographic Services, Department of Fisheries and Oceans. Campbellton is referred to as a secondary tidal port where tidal information is given in relation to the primary tidal port at Quanset Pierre on the eastern coast of the Gaspé Peninsula. Relevant tide characteristics are summarized in Table 2.2.

Background information describing climatic characteristics of the Walker Brook watershed is available from the Atmospheric Environment Service and other agencies such as Environment New Brunswick. Meteorologic data are measured at several locations within the study area as shown on Figure 1.2. The data collected is comprised mainly of daily precipitation and temperature records; however, hourly measurements are also available for first order stations located in Campbellton and Charlo, for which a total of 24 years of data is available to date. A summary of monthly average temperature and precipitation characteristics for Campbellton is given in Table 2.3. Additional information on rainfall intensities and frequency analysis are discussed in Appendix B. Additional climatic data are available in the vicinity of the study area at Balmoral and Tide Head (see Appendix B).

In addition, snow course measurements are collected by the New Brunswick Department of the Environment (NBDOE, 1981, 1982), at several locations within the study area, as indicated on Figure 1.2. To date, snow course data have been collected for the winters of 1980-1981 and 1981-1982. Snow course data are collected approximately every two weeks during the months January to March. A summary of available snow cover data for the Walker Brook watershed is presented in Table 2.4. Snow cover amounts are highly variable but are frequently excess of 80 cm depth.

Some information on ice cover characteristics along the Brook have been measured by the New Brunswick Department of the Environment. The available information covers part of the winter of 1981 and the winter of 1982 and is summarized in Table 2.5. Typical ice thickness has been observed in the range of 25-45 cm of ice cover on the Brook.

TABLE 2.1
Summary of Available Discharge Measurements
(a) Walker Brook Station 01BJ009
(b) Eel River

	<u>Jan</u>	<u>Feb</u>	<u>Mar</u>	<u>Apr</u>	<u>May</u>	<u>Jun</u>	<u>Jul</u>	<u>Aug</u>	<u>Sept</u>	<u>Oct</u>	<u>Nov</u>	<u>Dec</u>	<u>Year</u>
Walker Brook (Station 01BJ009)													
Period of Record : 1980 - 1981													
Mean Discharge (m ³ /s)	.221	.609	.818	4.52	3.81	2.54	.865	.414	.296	1.88	1.00	2.76	1.71
Max. Daily Discharge (m ³ /s)	.312	1.10	1.60	8.94	7.28	4.55	2.14	1.37	.872	4.00	2.55	14.4	14.4
Eel River (Station 01BJ004)													
Period of Record : 1967 - 1981													
Mean Discharge (m ³ /s)	.436	.430	.709	5.13	1.04	1.73	.707	.548	.915	1.42	1.24	.958	2.06
Max. Daily Discharge (m ³ /s)	2.55	2.32	12.5	41.1	47.3	10.3	6.63	13.4	24.0	8.47	6.26	2.00	47.3

Source: Surface Water Data, Atlantic Provinces
Inland Waters Directorate
Water Resources Branch
Water Survey of Canada
Environment Canada
1980-1981

TABLE 2.2
Tide Characteristics

	<u>Primary Tide Gauge Pointe St. Pierre (m)</u>	<u>Secondary Tide Gauge Campbellton (m)</u>
Mean High Tide	1.3	1.1
Maximum High Tide	1.7	1.9
Maximum Recorded Tide	2.3	2.4
Reference Station/		
Index Number	2310	2175

- Source : 1) Canadian Tide & Current Tables
Volume 2
Gulf of St. Lawrence
Fisheries and Oceans Canada
1982
2) Tides, Currents & Water Levels Division
Canadian Hydrographic Service
Fisheries and Oceans Canada

Note: Elevations expressed are related to Geodetic Survey of Canada datum.
Tide data at the secondary gauge are evaluated from those available at
the primary gauge.

TABLE 2.3

Summary of Temperature and Precipitation

Station: Campbellton

	<u>Jan</u>	<u>Feb</u>	<u>Mar</u>	<u>Apr</u>	<u>May</u>	<u>Jun</u>	<u>Jul</u>	<u>Aug</u>	<u>Sept</u>	<u>Oct</u>	<u>Nov</u>	<u>Dec</u>	<u>Year</u>
Mean Temperature (°C)	-11.4	-10.3	-4.3	2.5	9.1	15.5	19.0	17.6	12.4	6.4	0.1	-7.9	4.1
Mean Max. Temperature (°C)	-6.3	-4.9	0.9	7.0	14.4	21.3	24.6	23.2	17.7	10.7	3.4	-3.8	9.0
Mean Min. Temperature (°C)	-16.5	-15.6	-9.4	-2.1	3.7	9.7	13.2	11.9	7.1	2.1	3.3	-12.0	-0.9
Years of Record	20	20	21	21	20	20	22	21	22	21	22	20	--
Mean Precipitation (mm)	99.7	83.0	90.0	63.9	76.2	80.5	87.5	101.6	88.0	84.3	96.0	99.3	1050.0
Mean Snowfall (cm)	87.5	72.9	73.4	26.9	2.5	0.0	0.0	0.0	0.0	5.5	33.0	84.0	386.5
Years of Record	20	20	21	21	20	21	22	22	22	21	22	20	--

NOTE: Period of Record based on date of station installation to 1980

SOURCE: Temperature and Precipitation, Atlantic Provinces, Canadian Climate, November 1951-1980, Atmospheric Environment Service, Environment Canada.

TABLE 2.4
Summary of Available Snow Cover Data
for the Walker Brook Area (1)

Station Name	Lat.	Long.	Day	January		Day	February		Day	March	
				Snow Depth (cm)	Water Equiv (mm)		Snow Depth (cm)	Water Equiv (mm)		Snow Depth (cm)	Water Equiv (mm)
Winter of 1980 - 1981											
Cold Brook	47°56'00"	66°40'00"	-	-	-	-	-	-	-	-	-
			13	79.0	111.8	4	55.9	137.2			
			20	71.6	129.5	10	85.4	127.0	3	30.7	114.3
			28	71.1	137.2	17	44.5	137.2			
McKendrick	47°56'55"	66°42'25"				24	31.8	106.7	21	44.7	119.4
			13	94.5	147.3	4	73.4	165.1			
			20	91.9	114.3	10	80.8	200.7			
			28	88.9	154.9	17	62.2	154.9	21	73.4	180.3
Sugarloaf	47°58'15"	66°41'00"				24	50.0	134.6			
			13	87.1	142.2	4	65.0	160.0			
			20	80.0	152.4	10	72.1	167.6			
			28	80.0	165.1	17	63.0	155.0	21	62.0	157.5
Campbellton	47°59'25"	66°40'40"				24	42.1	129.5			
			4	54.6	137.2	3	37.6	114.3			
			14	85.9	129.5	10	62.2	149.9			
			20	68.6	132.1	17	45.2	116.8			
Duffs Lake	47°58'40"	66°44'50"	28	69.1	142.2	24	37.3	116.8	21	51.8	127
			4	60.2	144.8						
			14	102.4	165.1	10	72.9	175.3			
			20	83.8	162.6	17	62.2	167.6			
Lily Lake	47°57'58"	66°38'45"	28	81.3	154.9	24	42.4	137.2	21	53.3	134.6
			-	-	-	-	-	-	-	-	-

1980-1981

Station			January			February			March		
Name	Lat.	Long.	Day	Snow Depth (cm)	Water Equiv (mm)	Day	Snow Depth (cm)	Water Equiv (mm)	Day	Snow Depth (cm)	Water Equiv (mm)
Winter of 1981 - 1982											
Cold Brook	47°56'00"	66°40'00"	3	41.5	31.8	13	79.5	165	13	95.0	234
			16	54.4	80.8	27	82.3	178	27	96.0	135
			30	60.7	82.6						
McKendrick	47°56'55"	66°42'25"	3	61.4	55.4	13	94.2	116.8	13	108.7	251
			16	76.7	81.3	27	93.5	114	27	90.9	119
			30	77.2	102						
Sugarloaf	47°58'15"	66°41'00"	3	48.3	30.5	13	78.5	155	5	98.4	137
			9	43.9	81.3	20	80.3	170	13	90.4	201
			16	59.4	63.0	27	88.6	163	20	89.4	269
			23	52.6	77.5				27	78.0	124
			30	63.8	106						
Campbellton	47°59'25"	66°40'40"	3	38.9	49.0	13	74.9	153	13	84.3	206
			16	49.3	54.6	27	83.6	147	27	73.9	109
			30	60.4	80.0						
Duffs Lake	47°58'40"	66°44'50"	3	48.3	51.6	13	86.4	157	13	89.9	226
			16	65.0	90.9	27	90.7	145	27	88.1	155
			30	65.3	90.0						
Lily Lake	-	-	13	52.3	89.9	13	96.1	187	13	109.0	302
			30	73.5	133	27	100.8	206	27	96.0	152

(1) Source (Environment N.B., 1981, 1982)

1981-1982

TABLE 2.5
Ice Cover Characteristics on Walker Brook (1)

Location	Date of Observation	Snow Cover Depth (cm)	Depth of Water (mm)	Ice Thickness (cm)	Comments and Observations
Campbellton	Jan. 7/81	18.3-82.3	61-152	28.9-45.7	N/A
Sugarloaf	Jan. 7/81	25.9-27.4	137-213	27.4-28.9	N/A
Campbellton	Jan. 10/81	1.5	61-213	21.3-30.5	N/A
Sugarloaf	Jan. 10/81	2.1	122-213	27.4-28.9	N/A
General	Jan. 3/82	N/A	N/A	Open	Shore ice only 0.3-1.2 m wide
Gallant	Jan. 9/82	N/A	N/A	5-13	Most of channel open; gauge locations frozen over
Gauge	Jan. 16/82	N/A	N/A	Partial	Gauge location frozen; rest of stream open
Sugarloaf	Jan. 23/82	5" crust	152	18.3-20.5	Downstream area frozen over
General	Jan. 30/82	N/A	N/A	N/A	Downstream frozen; open areas near Val D'Amour
Gallant	Jan. 23/82	N/A	N/A	18.3-42.7	Deep snow on shore, windswept ice
Sugarloaf	Feb. 5/82	25.4-30.5	51-152	20.3-33	Stream frozen to river bed in some locations
Gallant	Feb. 5/82	18-20	152	43-51	Open water near banks due to deep snow insulation
Gauge	Feb. 5/82	23	127	10	Open water just upstream (-15°C)
Gauge	Feb. 13/82	20-31	N/A	N/A	Open just upstream, and open in Val D'Amour area
Gallant	Feb. 20/82	23	0-100	20-48	Frozen to bottom in some areas
Sugarloaf	Feb. 20/82	28-31	0-152	25-43	Open water visible both u/s and d/s
Gauge	Feb. 20/82	33	N/A	N/A	Open water just upstream
Gauge	Feb. 27/82	N/A	N/A	N/A	Open water u/s and Val D'Amour area
Gauge	Mar. 5/82	15-46	0-76	10	No open water visible near gauge
Sugarloaf	Mar. 5/82	33-46	0-127	28-46	Open water visible both u/s and d/s
Gallant	Mar. 5/82	20-38	0-127	25-46	Frozen to bed in some areas
Gauge	Mar. 13/82	N/A	N/A	Frozen	Large areas open due to warm weather
Gallant	Mar. 20/82	5	0-178	0-46	Channel partly open
Sugarloaf	Mar. 20/82	31	0-127	25-41	Large areas of stream were open
Gauge	Mar. 20/82	N/A	N/A	N/A	Open water u/s and d/s of gauge
Gauge	Mar. 27/82	N/A	N/A	N/A	Open water along channel

(1) Source (NBDE, 1981-82)

3.0 HYDROLOGIC ANALYSES

3.1 General

The 1:20 and 1:100 year recurrence interval peak flows were determined for Walker Brook using alternative estimating techniques:

- 1) Statistical Analyses
 - Regional Flood Frequency Analysis
 - Single station transfer from nearby watershed
- 2) Deterministic Analysis - Instantaneous Unit Hydrograph technique

These alternative estimating techniques were used for comparison purposes and in order to assess the reliability and accuracy of the available peak flow estimates.

The Regional Flood Frequency Analysis was used to augment the existing data base at the hydrometric station on Walker Brook since the period of record at this location is too short for any reliable statistical analyses. The transfer of peak flow estimates from a nearby watershed with a more complete record also attempts to overcome the short period of record on Walker Brook.

The deterministic computer modelling technique (more specifically, the HYMO model), was deemed necessary due to possible limitations imposed by the Regional Flood Frequency Analysis. These limitations are mainly due to the relatively small drainage area and the steep terrain encountered in the Walker Brook watershed (Acres, 1977).

The following sections outline the procedures used in the development and application of these techniques for estimating peak discharges associated with the 1:20 and 1:100 year recurrence interval flood events.

3.2 Statistical Analyses

3.2.1 Introduction

The Water Survey of Canada hydrometric station (No. 01BJ009) located on Walker Brook is a continuous type recorder which began operation on October 11, 1980. The available period of record of this station is, therefore, too short for undertaking a reliable statistical analysis of peak discharge. For this reason, the results of a Regional Flood Frequency Analysis prepared for the Canada-New Brunswick Flood Damage Reduction Program were initially used to determine peak flow conditions (Acres, 1977).

Some refinements to peak flow estimates were then made by transferring a statistical analysis using a suitable period of record from a similar watercourse in close proximity to Walker Brook.

3.2.2 Regional Flood Flow Estimates

The applicable equations for Northern New Brunswick are presented below and utilize physiographic parameters of Walker Brook to predict the 1:20 and 1:100 year peak flows (Acres, 1977).

$$QP_{20} = 1.12 A^{.861} W^{5.47} FDS^{-.920} (+48\%, -32\%) \quad (3.1)$$

$$QP_{100} = 1.48 A^{.865} W^{6.42} FDS^{-1.08} (+60\%, -37\%) \quad (3.2)$$

(95% confidence limits shown in brackets)

where: A = drainage area (km²)

W = (A - A_s)/A

A_s = area swamps and lakes (km²)

FDS = proportion of annual maximum floods occurring in March, April, May period at the drainage basin centroid

Table 3.1 lists the ranges associated with the independent variables used in the development of equations 3.1 and 3.2. Table 3.1 also shows the values of the independent variables determined for Walker Brook. Table 3.2 summarizes the peak flows estimated by applying equations 3.1 and 3.2 using the data summarized in Table 3.1.

With reference to Table 3.1 it is evident that in some instances (slope, area in the upper reaches), the value of the independent variables for Walker Brook are just outside of the range recommended for application of Equations 3.1 and 3.2. It can also be seen from the 95% confidence limits which accompany the equations that the range of flows associated with peak flow estimates using the Regional Analysis is quite substantial. For these reasons, it was felt that application of the results of the Regional Analysis presented in Table 2.2 without confirmation by secondary estimates could be somewhat questionable. It was, therefore, concluded that verification of these flow estimates by secondary techniques was required in order to confirm the level of accuracy of the 1:20 and 1:100 year flood estimates. Two additional independent techniques were used to evaluate flow estimates. The first of these was a transfer of peak flow estimates by a single station frequency analysis (Section 3.2.4) and the second was application of a deterministic modelling technique (Section 3.3).

3.2.3 Single Station Statistical Analysis of Eel River Flood Peaks

A secondary check for the 1:20 and 1:100 year peak flow given by the Regional Flood Frequency Analysis was made by undertaking a statistical analysis of peak flows at a longer term hydrometric station on a nearby watercourse with similar hydrologic characteristics. A review of available information led to the use of the hydrometric station located on the Eel River near Eel River Crossing (Sta. No. 01BJ004). This gauge is located on the main branch of the Eel River and has been in operation from 1967 to date. The gauge is a continuous type recorder and measures discharge from a watershed area of some 88.6 km² (compared to a drainage area of 67.4 km² for Walker Brook at the outlet).

The Eel River gauge was selected for use in this analysis as it is in close proximity to the Walker Brook watershed, and because of similar topography, land use and hydrologic characteristics. The meteorological conditions of the two watersheds are also considered to be similar and the drainage areas at the gauge locations are similar. Table 3.3 presents a summary comparison of several hydrologic characteristics determined for each watershed.

i) Testing of the Data Base

A major assumption in flood frequency analysis is that the sample set of data is a random sample from a single (unknown) probability distribution. The assumption of randomness can be disproved by some feature of a non-random nature such as persistence or trend (serial correlation). For this reason, the sample data set of annual maximum instantaneous discharge was tested for stationarity and randomness using graphical techniques. Additional details on these analyses are given in Appendix A.

From the results of these analyses it was evident that the extreme flow series can be regarded to be independent and free of trend and persistence. Therefore, it was concluded that the subsequent frequency analysis applied to the Eel River data can be undertaken using the available data base and the annual maximum flow series.

ii) Selection of Distribution

Several theoretical flood frequency distributions are available for use in fitting the peak discharge data series. The computer program, FDRPFFA (Environment Canada, 1979), provided by the Water Planning and Management Branch, Environment Canada, was utilized for computing the 1:20 and 1:100 year flood peaks and the corresponding 95% confidence limits.

In flood frequency analysis, it is not possible to pre-determine which frequency distribution should be used. The selection of the appropriate distribution for use in this investigation was based on comparisons of the computed "T" year flood using the methods as outlined in the FDRPFFA program (Environment Canada, 1979) and summarized as follows:

- comparison of various fitting techniques to the regional peak flow estimates using available regional techniques
- comparison of the theoretical statistics of skew and kurtosis to those estimated from the station data
- visual comparison of the degree of fit of each distribution to the available data set
- estimation of upper limit of flow estimates.

The frequency analysis results for Eel River data are described in detail in Appendix A and summarized in Table 3.4. The Log Pearson Type 3 (LP3) and Three Parameter Log Normal (3PLN) Distributions were both found to give an acceptable fit to the available data base (see Appendix A). The LP3 distribution was selected for the purposes of this investigation since the flood estimates (for 1:20 and 1:100 year peak flows) were more conservative than the log normal distribution (recommended for the North Region of New Brunswick (Acres, 1977)) and are slightly more accurate when comparing the standard error of the two 3 parameter distributions. The use of the LP3 distribution provides intermediate flood estimates which are slightly lower than the estimates by the 3PLN. (On the other hand, the general conclusions drawn by comparison to the regional flood estimates would not change if the 3PLN was used for comparison purposes.) Based on the LP3 distribution, the estimated 1:20 and 1:100 year instantaneous peak flows on the Eel River at the gauge location were found to be 56.5 m³/sec and 72.1 m³/sec respectively.

3.2.4 Flow Transfer - Eel River to Walker Brook

When measured streamflow data necessary to determine flood levels are either inadequate or not available, a flow transfer technique can sometimes be used to eliminate major sampling errors associated with short-term records, or to synthesize information at ungauged sites. Furthermore, if a hydrometric station with sufficient data is available in the vicinity of the point at which design flood peaks are required, it is recommended that use be made of that hydrometric data.

The extrapolation of the measured peak flows for the nearby Eel River to the study area was accomplished in the following manner:

- a) The LP3 distribution was fitted to the sample series of annual maximum floods available for the Eel River.
- b) The 1:20 and 1:100 year peak discharge values were estimated by using the LP3 Distribution
- c) The 1:20 and 1:100 year flood peaks at the gauged and the design sites were then calculated using the regional technique (Acres, 1977) by applying equations 3.1 and 3.2
- d) The ratio of the calculated flood peaks at the two sites was then used to convert the gauged values to the desired design flood peaks, by applying the following equation:

$$Q_T = Q'_T (QC_T/QC'_T) \quad (3.3)$$

where Q_T = desired design flood peak at selected locations in the study area for return period T-years (m^3/s)

Q'_T = flood peak of return period T obtained from flood frequency curve (m^3/s) on Eel River

QC_T & QC'_T = flood peaks calculated using regional technique at the location of interest and the gauge site respectively

- e) It is assumed that the standard errors of estimates of the Q'_T values apply to the design Q_T values.

Peak flows calculated for the Eel River at the gauge are summarized in Table 3.4. The results of the Regional analysis for Walker Brook given in Table 3.2 were used to calculate the transfer ratio by Equation 3.3. Based on the analysis, the estimated 1:20 and 1:100 year peak flow rates for Walker Brook are presented in Table 3.5.

A comparison of peak flow estimates in Tables 3.2 and 3.5 confirms that the Regional Flood Frequency Analysis produces acceptable results at the outlet of Walker Brook.

3.2.5 Summary of Statistical Analyses

The following points summarize the main findings and conclusions of the statistical analysis:

- 1) The available peak flow record for the Eel River is free of trend and persistence and it is, therefore possible to undertake a standard frequency analysis using the available data base.
- 2) The two, three-parameter distributions (3PLN and LP3) were found to give an acceptable degree of fit to the available data.
- 3) Comparing the 3PLN and LP3 distributions, the 1:20 and 1:100 year peak flow estimates by the LP3 had smaller standard errors. The use of LP3 distribution tends to confirm peak flows by the Regional technique for the Eel River.
- 4) The 1:100 and 1:20 year maximum instantaneous discharge for the Eel River by the various techniques is given in Table 3.4 for the gauge location.
- 5) The 1:100 and 1:20 year maximum instantaneous discharge for the outlet of the study area by transfer of Eel River estimates are summarized in Table 3.5.
- 6) The single station frequency analysis confirms that the Regional Flood Frequency Analysis can provide reasonably accurate estimates of the 1:20 and 1:100 year flood peaks on the Eel River. It is, therefore, concluded that the use of the Regional Flood Frequency Technique on watersheds with similar flood characteristics in Northern New Brunswick would also result in reasonable estimates for 1:20 and 1:100 year floods (See Table 3.2 for summary of regional results for Walker Brook).

3.3 Deterministic Analyses

3.3.1 Introduction

The 1:20 and 1:100 year peak flows for the Walker Brook watershed were also simulated by using a synthetic unit hydrograph procedure known as HYMO (U.S. Dept. of Agric., 1973). The input requirements of this hydrologic simulation technique include both meteorological (rainfall/snowmelt) data and physio-

graphic characteristics (land use, time to peak values, constituent soil characteristics, etc.) of the study area.

The following sections describe the hydrologic procedures used in the development and application of the HYMO model in the preliminary determination of design flow estimates for the Walker Brook watershed. A comparison of the deterministic and statistical estimates is then given.

3.3.2 HYMO Model Structure

The program "HYMO" is a hydrologic computer model used to simulate the surface runoff from a particular drainage area for a specific meteorological input. For transformation of the input into runoff hydrographs, the program uses a synthetic unit hydrograph technique and the Soil Conservation Service rainfall-runoff relationships (SCS, 1972).

The program generates a hydrograph for each selected sub-drainage area of the watershed. The hydrographs, beginning at the upstream end of the basin, are added together and routed downstream until the storm event hydrograph at the outlet of the entire watershed is obtained.

The routing of the simulated hydrograph requires as input, characteristics of the channel and floodplain cross-sections, hydraulic roughness coefficients, and reach slopes and lengths. In order to refine the flood routing computations, field surveyed cross-sections were input to the model to obtain an accurate representation of the channel routing characteristics.

The reservoir routing effects were also modelled using the HYMO program. The input requirements are discharge/storage relationships for each reservoir. Field measurements and available topographic maps were used to obtain these relationships for each of the main water supply reservoirs. The reservoirs modelled are Pritchard Lake, Smith Lake and the large unnamed lake immediately downstream of Smith Lake, since these are the only impoundments in the watershed.

In order to simulate the hydrologic response of the Walker Brook watershed, and to provide distributed flow estimates, the area was divided into 12 sub-catchments which included 7 channel routings and 3 reservoir routings. In doing so, inflows and outflows from the lakes and reservoirs could be modelled directly and distributed peak flow estimates could also be determined along the study reach.

The watershed discretization (see Figure 3.1) was established through examination and interpretation of existing 1:10,000 scale mapping of the study area. This mapping was used to measure sub-drainage areas, slopes and land-use characteristics, etc. Figure 3-2 illustrates, in schematic form, the modelling breakdown applied in this analysis.

3.3.3 Meteorologic Data

The climate of the Walker Brook watershed could be characterized as humid continental with severe winters and cool summers. The mean annual precipitation is about 1000 mm, with a fairly even monthly distribution (see also Table 2.3). Flood conditions in the watershed are generated as a result of snowmelt and/or rain on snow conditions. However, severe rainstorm events can also result in extreme flow conditions.

1) Rainfall

The design storm pattern used in the calculations was based on the Atmospheric Environment Services (AES, 1980) interpretation of 6-12 hr. rainstorms in the Maritime Provinces. This temporal distribution was used as it best represents typical rainstorm patterns which occur in New Brunswick. The temporal distribution of the design storms used is presented in Table 3.6. The 12-hour AES temporal distribution was used based on an analysis of the time of concentration of the Walker Brook watershed. From recorded events, it was determined that the time of concentration for the Walker Brook watershed at the outlet is approximately 6-8 hours. This was substantiated by applying various empirical equations which yielded comparable values for time of concentration. Flood simulations using 6-hour and 24-hour storm distributions resulted in lower peak discharge values which also confirms the use of a 12-hour storm distribution.

The total point rainfall was obtained from Rainfall-Intensity-Duration (RID) curves derived for the Campbellton meteorologic station. The resulting total rainfall for the 1:100 and 1:20 year 12-hour storms was found to be 85 mm and 67 mm respectively (based on 19 years of data). An analysis of available meteorologic data in the region has revealed a high variability in the T-year rainfall estimates. The variability is attributed to the relatively short periods of record available. For example, the total rainfall for the upper 95% confidence limits is estimated to be 109 mm and 84 mm for the 1:100 and 1:20 year storms respectively. (Refer to additional details on discussion of rainfall variability in Appendix B.)

ii) Snowmelt

Based on an analysis of discharge records for nearby hydrometric stations, the majority of peak flow events occur in April and May. Peak flow events occurring in April are likely the result of snowmelt and rain on snow conditions. (As indicated in Table 3.3, the average snow on the ground at the end of March is about 500 mm, and April averages about 100 degree days of thawing). Peak flow conditions during the month of May are likely due to a combination of saturated ground conditions and rainfall events.

An analysis of typical snowmelt conditions was undertaken in order to provide some insight into the relative importance of spring and summer flood conditions. The rainfall plus snowmelt for various recurrence intervals was established by analysis of available data for the Campbellton meteorological station. This analysis uses actual recorded meteorologic data to obtain the 1 to 10 day melt plus rainfall totals available for runoff. The algorithm used was derived from an energy budget method developed by the U.S. Army Corps of Engineers for heavily forested areas. Using this algorithm and recorded meteorologic data from Campbellton, the 1:100 year and 1:20 year 24-hour melt plus rain are about 57 mm and 43 mm respectively. (Additional details on the snowmelt Algorithm are given in Appendix B).

A secondary analysis using an energy budget equation was also undertaken utilizing the theoretical energy inputs for the spring melt period (April/May)

for the Campbellton meteorological station. This secondary check confirmed the snowmelt estimates by the procedure previously discussed. The results of this secondary analysis are presented in Appendix B.

3.3.4 Hydrologic Input Parameters

i) Time to Peak and Recession Constant

In order to simulate the response of a specific watershed to a rainfall event using the HYMO procedures, two unit hydrograph parameters must first be established. They are the time to peak and the recession constant for the unit hydrograph. To facilitate calibration of these parameters to the available streamflow records on the watercourse, both parameters were first derived using various empirical equations (See Appendix B for details). However, since the available empirical equations were not derived for Canadian conditions, it is necessary to modify the unit hydrograph parameters to reflect local runoff characteristics. These initial estimates of unit hydrograph characteristics were modified by calibration of the HYMO model. The parameter applicability was then further tested through subsequent model validation simulations (see section 3.3.5).

A summary of the relevant physiographic and unit hydrograph parameters which were determined for each subwatershed are given in Table 3.7.

ii) Soils

The soil types were derived from a soil survey covering the Walker Brook watershed (ARDA, 19). The soils are mainly comprised of the Balmoral and Victoria series (see Table 3.8). The Balmoral series is one of the Ablational tills and consists of gravely clay loams. The Victoria series is from glacial fluvial drifts and consists of well mixed gravely sandy loams. However, the fact that the soil cover above the bedrock in many locations is either very shallow or essentially non-existent has a major influence on the water retention capacity of the soils.

iii) Land Use

The Walker Brook watershed is primarily forest covered. A "coefficient of overburden" was previously mapped by Acres for Northern New Brunswick. By interpretation of this information the assumption was made that exposed bedrock comprised about 25% of the watershed area. In the lower reaches of Walker Brook, the primary land use is residential.

iv) Soil Cover Complex Number

A runoff index factor combining the hydrologic soil group and land use characteristics is referred to as a hydrologic soil cover complex number, CN. Simulated runoff volumes are proportional to the complex number according to the following rainfall-runoff relationship:

$$Q = \frac{(P - 0.2s)^2}{P + 0.8s} \quad (3.4)$$

where Q = amount of runoff (inches or mm)

P = precipitation amount (inches or mm)

$s = \frac{1000}{CN} - 10$

Using the available soils land use information, average soil cover complex numbers were determined for each of the 12 sub-basins (see Appendix B for details of procedures for estimating soil cover complex numbers). The average soil cover complex numbers are summarized in Table 3.7.

3.3.5 Model Calibration and Validation

Utilizing the available meteorological data from recording stations in the area and the streamflow records from the Walker Brook station, the HYMO model was calibrated to summer storm flood events and spring melt events.

The general procedures for calibration of the HYMO model are summarized as follows:

- 1) isolate suitable calibration and validation events from available meteorologic and hydrologic data
- 2) from meteorologic data, select a storm which is representative of the associated flood conditions under consideration
- 3) from hydrologic data determine the average watershed runoff volume for the storm under consideration
- 4) using the total rainfall amount and the average runoff volume, compute the average curve number for the entire watershed
- 5) using the average watershed curve number, compute the individual sub-watershed curve numbers
- 6) vary the unit hydrograph parameters K and T_p for each subwatershed to change the shape of the computed outflow hydrograph
- 7) compare computed outflow hydrograph with recorded outflow hydrograph
- 8) iterate model parameter adjustments until a good fit is obtained.

From a review of the meteorologic and hydrometric data available for Walker Brook, available flood events were selected for calibration and validation purposes.

The first event was a summer rainfall event which occurred August 17-18, 1981. During the 7 hours of the storm, 37.6 mm of rain fell on the basin. The recorded peak discharge was 2.15 cms and occurred roughly 8 hours after occurrence of the peak rainfall.

The second event used was a peak flow which occurred as a result of rain and snowmelt on April 8-10, 1981. From temperature and rainfall records, it was estimated that 91 mm of rain plus melt occurred over the basin. The recorded peak discharge occurred at 1800 hours on April 10 and had a magnitude of 10.7 cms.

The unit hydrograph parameters which resulted in the "best" comparison of simulated and recorded hydrographs are summarized in Table 3.7.

A graphical comparison of simulated and recorded hydrographs is presented in Figure 3.3. While these are the best results that could be achieved with the available data base, it is evident that the results are not completely

satisfactory. While the time to peak and hydrograph timing factors are adequately represented, the peak flow is simulated with some error. Additional events of significant magnitude are required to improve the model simulation accuracy. Based on experience in previous investigations, it is usually necessary to calibrate hydrologic models using events which produce flows of the same magnitude as the design events (1:20, 1:100). With reference to the Regional Analysis and annual discharge summaries, it was evident that no such events exist in the present Walker Brook data base. Therefore, until such time as additional peak flow data becomes available, the HYMO model cannot be completely calibrated and validated for application to the watershed.

3.3.6 Peak Flow and Sensitivity Analysis

Using the results of the preliminary HYMO calibration the 1:100 year and the 1:20 year rainfall and rainfall plus snowmelt events were simulated. The results are presented in Table 3.9. Comparisons with Table 3.2 and 3.5 will indicate that the peak flows simulated by HYMO compare fairly well with the statistical estimates. In addition, it is evident that both summer rainstorms and snowmelt events can result in the occurrence of significant flood events.

Because of the variations in design storm depths, historic events were also simulated with the HYMO model. The historic events considered for modelling were Hurricane Belle and the storm of August, 1981. Hurricane Belle occurred during August, 1976 and was centred near Edmunston. The storm of August, 1981 was centred near Plaster Rock and although not a hurricane, resulted in higher rainfall amounts than Hurricane Belle. Because a detailed examination of the August, 1981 storm is not currently available, the storm could not be simulated accurately with the HYMO model. However, a detailed examination of Hurricane Belle has been undertaken and this event was modelled, assuming this event was centred over the Walker Brook watershed, resulting in a peak flow rate of $104.1 \text{ m}^3/\text{s}$. Reference to the summary of peak flow estimates given in Table 3.11 will reveal that estimated peak flows associated with the occurrence of Hurricane Belle over the watershed are greater than the peak flows by the Regional Flood Frequency analysis at the upper 95% confidence limit.

The impact of significantly higher rainstorm amounts on peak flow estimates was also analysed as part of the sensitivity testing. The upper limit for the probable maximum precipitation event (PMP) for the study area in a 24-hour period was determined to be about 390 mm (Atmospheric Environment Service, 1981). This is about four times the 1:100 year rainstorm determined for the study area (see Appendix B), and is assumed to be the upper limit for precipitation in the study area. The resulting peak discharge rates for Walker Brook at the outlet were found to be 346.3 and 377.7 m³/s, assuming partly saturated and saturated antecedent soil moisture conditions (AMC II and AMC III) respectively. Based on the present HYMO model, the flood peak corresponding to the PMP is over four times the magnitude of the 1:100 year flood peak estimated by the Regional Method (refer to peak flow summary in Table 3.11). This comparison reflects the sensitivity of peak flow estimates to extreme precipitation events which could occur over the Walker Brook watershed.

Since it was not possible to finalize calibration of the model, the sensitivity to variations in model input was evaluated by varying the input parameters $\pm 20\%$. For example, the change in time to peak was about ± 1 hour. The results of this analysis are presented in Table 3.10 and reveal that for the 1:100 year storm, varying the input parameters $\pm 20\%$ constitutes a $\pm 7.5\%$ change in peak discharge rates at the outlet.

The influence of upstream storage was also examined by assuming that the water supply reservoirs are at a drawdown stage when the design storms begin. The drawdown stage was assumed to be 0.60 m below that of normal summer water levels. (The maximum average drawdown in the water supply reservoirs is somewhat larger and occurs in the fall. However because the primary purpose of the reservoirs is water supply and not flood control, a 0.60 m drawdown was assumed to be typical for the purposes of this analysis.) The results of this analysis indicate that peak flows for the 100 year design storm are reduced less than 2% (Table 3.10). From these results it can be seen that lowering the water supply reservoirs prior to a severe storm has minimal effect on the peak flow rates on the main branch of Walker Brook. This is mainly due to the small contributing area associated with the water supply reservoirs.

There is no provision for operating the reservoirs (i.e. drawing down prior to runoff event) for flood control. Therefore, it is reasonable to derive peak flows without accounting for the possible effects of upstream storage (which would be small in any case).

Since the reservoirs are not operated for flood control and since the available storage does not significantly affect peak flows, it is reasonable to estimate the 1:20 and 1:100 year peak discharge rates by statistical techniques.

3.3.7 Summary of Deterministic Analyses

- 1) Upon review of the meteorologic and hydrometric data available for Walker Brook, two events were selected for calibration. However, until more data becomes available, satisfactory calibration and validation of the HYMO model cannot be attained.
- 2) Using the results of the preliminary HYMO model calibration, the peak flows were simulated for summer rainstorm and spring melt events. Table 3.9 summarizes the results of this analysis.
- 3) It is evident from Table 3.9 that for the Walker Brook watershed, summer rainstorms can result in significant flood events. This can be attributed to the relatively small drainage area and the higher incremental intensities associated with summer rainfall events. The rainfall plus snowmelt analyses resulted in flow estimates slightly smaller than the summer rainstorms. This is consistent with findings in previous studies for small watersheds.
- 4) It was found that the 1:20 and 1:100 year design flows simulated independently by the HYMO model supported the peak flow estimates derived for the watershed by the Regional Analysis for Walker Brook.
- 5) Furthermore, it was found that a good comparison of HYMO peaks and Regional Estimates extends to the upper reaches of Walker Brook where the drainage areas are less than that recommended for use in the

Regional Analysis (Acres, 1977). This also supports the use of the regional analysis for estimating peak flows in the watershed.

- 6) It was found that water supply reservoirs in the watershed have a negligible effect on flood peaks.
- 7) Based on the present HYMO model, the flood peak corresponding to the probable maximum precipitation event (PMP) is over four times the magnitude of the upper confidence limit of the 1:100 year flood peak estimated by the Regional Method (see Table 3.11).
- 8) The centering of historical storms, which occurred in recent years in other parts of New Brunswick over the Walker Brook watershed would result in flood peaks of the same order of magnitude as the 1:100 year upper 95% confidence limit peak flows estimated by the Regional Flood Frequency Analysis.
- 9) The 1:20 and 1:100 year design events centred over the Walker Brook watershed would result in peak flows of a magnitude comparable to the statistical estimates. However, some evidence suggests that the total rainfall associated with the 1:20 and 1:100 year rainstorms could be higher than that determined from a statistical analysis of records at Campbellton and Charlo (see Appendix B).

3.4 Main Conclusions and Recommendations of Hydrologic Analyses

Table 3.11 presents a brief summary comparison of peak flows estimated by various techniques on Walker Brook. The main findings and conclusions of the hydrologic analyses are summarized as follows:

- 1) The comparison of single station estimates (by transfer of flood peaks from the Eel River to Walker Brook) confirmed that the flood peaks estimated by the Regional Flood Frequency analysis are reasonable values which are appropriate for use in the Walker Brook watershed.

- 2) The available data base was adequate for only a preliminary calibration of the HYMO model. Additional meteorological and discharge data is required for further calibration and verification of deterministic precipitation-runoff models such as HYMO. However, the HYMO simulations have confirmed that peak flow estimates by the Flood Frequency Analysis are reasonable estimates.
- 3) Walker Brook was found to have a smaller drainage area and a steeper basin slope than nearby gauged watersheds (see Table 3.3). These parameters, combined with other watershed characteristics (e.g. high snow accumulation and melt, and surficial soil types, etc.) indicate that the watershed could generate higher peak flow rates than other watersheds of the same size in the region. In addition, it was found that late summer/fall hurricanes could produce higher runoff than spring rainfall/snowmelt events for the watershed. These factors and others are not explicitly accounted for by utilizing the "mean estimates" of the Regional Flood Frequency technique. Therefore, in order to provide conservative, yet realistic peak flow estimates, taking into account the flood producing potential of the Walker Brook watershed, it is recommended that the 1:20 and 1:100 year peak flow estimates associated with the upper 95% confidence limits by the Regional Flood Frequency Analysis should be adopted. Therefore, the 1:20 and 1:100 year peak flow estimates used in the subsequent hydraulic analyses are as follows (see also Table 3.2):

	<u>1:20</u>	<u>1:100</u>
	<u>U.C.L.</u>	<u>U.C.L.</u>
Walker Brook near outlet	57.9	82.9
Walker Brook near Sugarloaf Mountain	53.4	76.5
Walker Brook Upstream from Sugarloaf Mountain	42.3	61.1

TABLE 3.1
Independent Variables for Application of
Regional Equations

	Area (km ²)	$\frac{(A - A_s)}{A}$ (proportion)	FDTs (proportion)	Slope (percent)
Applicable Parameter Range	25-5000	0.85 - 1.0	.45 - 1.00	.08 - 1.57
Walker Brook near outlet	67.4	.99	1.00	1.79
Walker Brook near Sugarloaf Mountain	61.9	.98	1.00	1.86
Walker Brook upstream from Sugarloaf Mountain	45.0	.99	1.00	1.98

TABLE 3.2
Regional Analysis Maximum Instantaneous Flow Rates

Location of Flow	Drainage Area km ²	1:20 yr Peak Flow cms		1:100 yr Peak Flow cms	
		Mean	U.C.L.	Mean	U.C.L.
Walker Brook near outlet	67.4 (69)	<u>39.1</u> (40)	57.9	<u>51.8</u> (53)	82.9
Walker Brook near Sugarloaf Mountain	61.9 (62)	<u>36.1</u> (36)	53.4	<u>47.8</u> (48)	76.5
Walker Brook upstream from Sugarloaf Mountain	45.0 (42)	<u>28.6</u> (27)	42.3	<u>38.2</u> (36)	61.1

Notes:

- (a) Bracketed values are Discharge Estimates by previous regional study (Acres, 1977)
- (b) Underlines are Discharge Estimates by applying equations 3.1 and 3.2 (present study)
- (c) Some refinements in drainage areas determined for the present study were possible due to the application of more accurate topographic mapping at a scale of 1:10,000 for the watershed; compared to 1:50,000 scale maps used in previous investigations
- (d) Mean : mean flood estimate by equations 3.1 and 3.2
U.C.L. : upper 95% confidence limit estimate by equations 3.1 and 3.2

TABLE 3.3
Comparison of Meteorologic and Physiographic Characteristics*
for Walker Brook and Eel River Watersheds

	<u>Walker Brook</u>	<u>Eel River</u>
Drainage Area - to gauge (km ²)	61.9	88.6**
- to outlet	67.4	185
Mean Annual Precipitation (mm)	1000	975
Median Snow Cover (Mar. 31) (mm)	500	475
Mean April Thawing Degree Days	100	100
Mean Annual Runoff (mm)	600	600
Average Catchment Slope (%)	1.79	1.14**
Coefficient of Overburden	1.5	2
1:20 year 24-hour rain (mm)	81.6	82.0
1:100 year 24-hour rain (mm)	100.5	106.7
% Forest Cover	60-80	40-80

* Source - (Acres, 1977) and (AES, 1981)

** at gauge

TABLE 3.4
 Summary Comparison of Peak Flow Estimates
on the Eel River at the Gauge Location

Description	Maximum Instantaneous Q (m ³ /s)					
	1:20 Yr.			1:100 Yr.		
	-95%	Mean	+95%	-95%	Mean	+95%
1) Regional Analyses (Acres DA = 91 km ²)	36.2	54	79.9	45.4	72.0	115.2
2) Regional Analyses (CCL DA = 88.6 km ²) (to gauge site)	35.8	52.6	77.8	44.4	70.5	112.8
3) Frequency Analyses (DA = 88.6 km ²) - LP3	38.5	56.5	74.5	35.3	72.1	108.9

TABLE 3.5
 Peak Flow Estimates by Transfer of
Eel River Frequency Analysis to Walker Brook

<u>Location of Flow</u>	<u>Drainage Area km²</u>	<u>1:20 yr Peak Flow cms</u>		<u>1:100 yr Peak Flow cms</u>	
		<u>Mean</u>	<u>U.C.L.</u>	<u>Mean</u>	<u>U.C.L.</u>
Walker Brook near outlet	67.4	42.0	55.4	53.0	80.0
Walker Brook near Sugarloaf Mountain	61.9	38.8	51.1	48.9	73.8
Walker Brook upstream from Sugarloaf Mountain	45.0	30.7	40.5	39.1	59.0

TABLE 3.6
Rainfall Distribution
for the 12-hour Design Storms (a)

<u>Time (hours)</u>	(b) 12-hour Design Storm <u>Distribution (%)</u>
0 - 1	3
1 - 2	7
2 - 3	10
3 - 4	11
4 - 5	12
5 - 6	12
6 - 7	13
7 - 8	9
8 - 9	13
9 - 10	6
10 - 11	3
11 - 12	1
	<u>100</u>

Notes:

- (a) Percent of total rainfall over 12-hour period.
The Rainfall distribution was derived by analysing a large number of actual rainstorm patterns for several stations in New Brunswick and the Maritimes.
- (b) Distribution given reflects the percentage of the total rainfall that falls over the period (Source AES, 1980).

TABLE 3.7
Physiographic and Unit Hydrograph Parameters for Sub-Watersheds

Basin Nos.	Area (km ²)	Length (km)	Weighted Slope (m/m)	Recession Parameter (hrs) (K)	Time to Peak Parameter (hrs) T _p	Average Soil Cover Complex Number (CN)
100	8.73	4.35	.025	2.65	2.72	78
101	5.31	4.60	.024	2.64	2.44	77
110	5.62	5.94	.028	2.51	2.47	70
120	5.60	5.49	.025	2.68	2.54	78
121	12.44	6.73	.035	2.37	2.90	75
102	7.33	5.60	.019	3.42	3.15	77
140	1.42	.66	.110	3.57	1.95	93
141	4.12	3.44	.010	4.71	3.12	66
142	4.04	3.25	.042	1.52	1.59	78
143	1.94	2.50	.033	1.59	1.34	74
103	5.47	6.08	.027	2.59	2.51	86
130	5.39	5.80	.038	1.96	2.11	71

TABLE 3.8
Summary of Hydrologic Soil Groups

<u>Predominant Soil Series and Type</u>	<u>Percent</u>	<u>Hydrologic Soil Group</u>
Victoria - well mixed gravel	18.6	AB
- sandy loam		
Balmoral - gravel clay loam	76.8	C
Maltais - gravel loam	4.6	CD

TABLE 3.9
Design Flows Generated by HYMO Model

<u>Location of Flow</u>	<u>Drainage Area km²</u>	<u>Peak Flow (cms)</u>			
		<u>1:20 yr. Spring</u>	<u>1:20 yr. Summer</u>	<u>1:100 yr. Spring</u>	<u>1:100 yr. Summer</u>
Walker Brook at outlet	67.4	25.6	43.1	36.5	66.9
Walker Brook at Sugarloaf Mountain	61.9	23.0	38.7	33.0	60.2
Walker Brook upstream from Sugarloaf Mountain	45.0	18.2	31.7	25.6	48.4

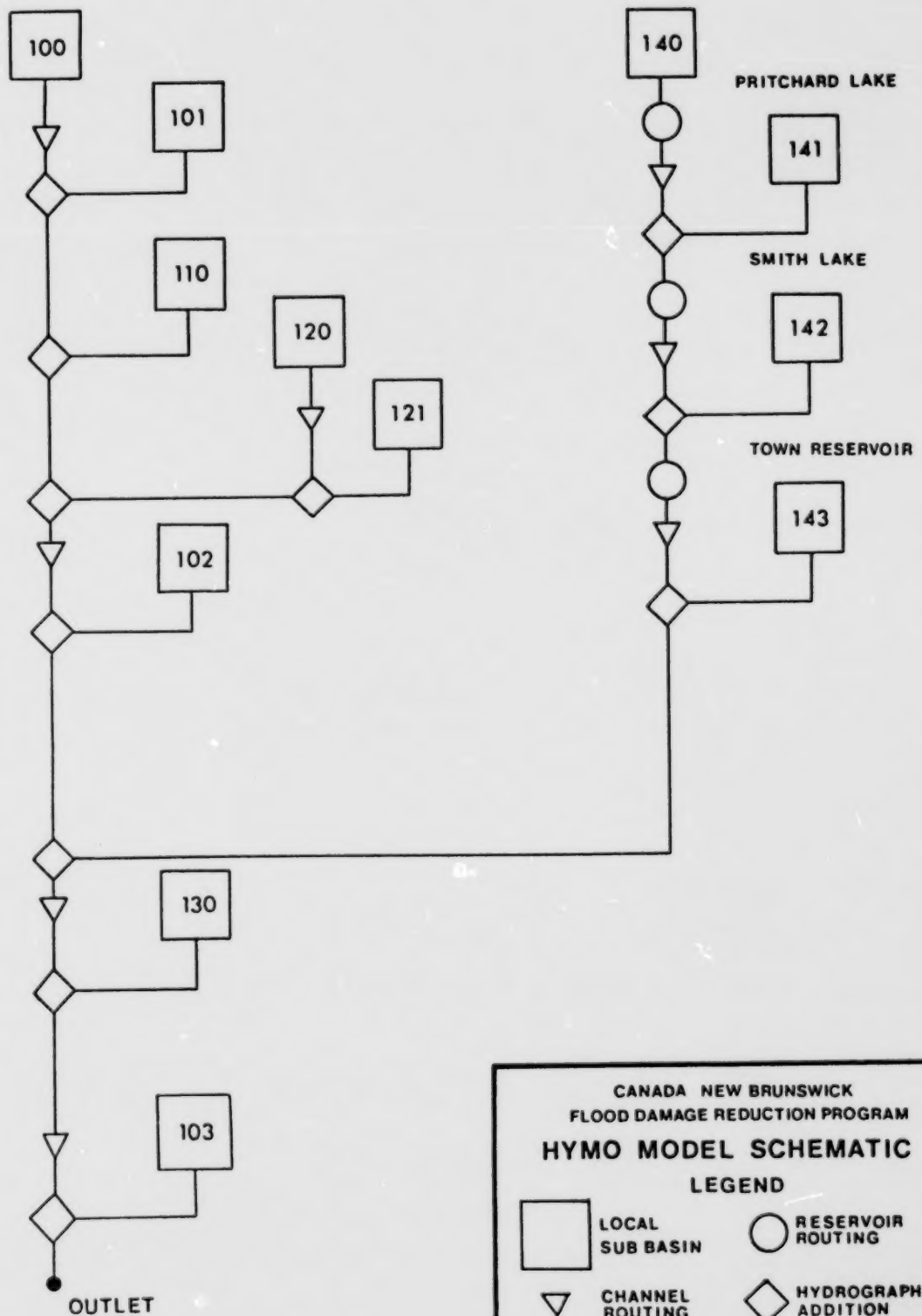
TABLE 3.10
Sensitivity Analysis of HYMO Model

<u>Changes to Model</u>	<u>Change in Peak Discharge at Outlet of Walker Brook (percent change in 100 yr. 12 hr. storm peak)</u>
Draw down water supply reservoirs 2 ft.	-2%
Increase k and T _p Parameters 20%	-7.5%
Decrease k and T _p Parameters 20%	+7.5%

TABLE 3.11
Summary Comparison of Peak Flow Estimates
Walker Brook near Outlet

Description	PMP and Historical Events	Maximum Instantaneous Q (m ³ /s)					
		1:20 Yr.			1:100 Yr.		
		-95%	Mean	+95%	-95%	Mean	+95%
a) <u>Statistical Estimates</u>							
1) Regional Analyses							
(Acres, DA = 69 km ²)	-	27.2	40.0	59.2	33.4	53.0	84.8
2) Regional Analyses							
(CCL, DA = 67.4 km ²)	-	26.6	39.1	57.9	32.6	51.8	82.9
3) Eel R. transferred							
(CCL, DA = 67.4 km ²)							
- LP3	-	28.6	42.0	55.4	26.0	53.0	80.0
b) <u>HYMO Estimates</u>							
1) 12-hr. storm	-	-	43.1	63.9	-	66.9	101.0
2) 24-hr. storm	-	-	41.2	**	-	58.2	**
3) Snowmelt + rain	-	-	25.6	-	-	36.5	
4) PMP - AMC III	377.7	-	-	-	-	-	-
- AMC II	346.3	-	-	-	-	-	-
5) Hurricane Belle	104.1	-	-	-	-	-	-

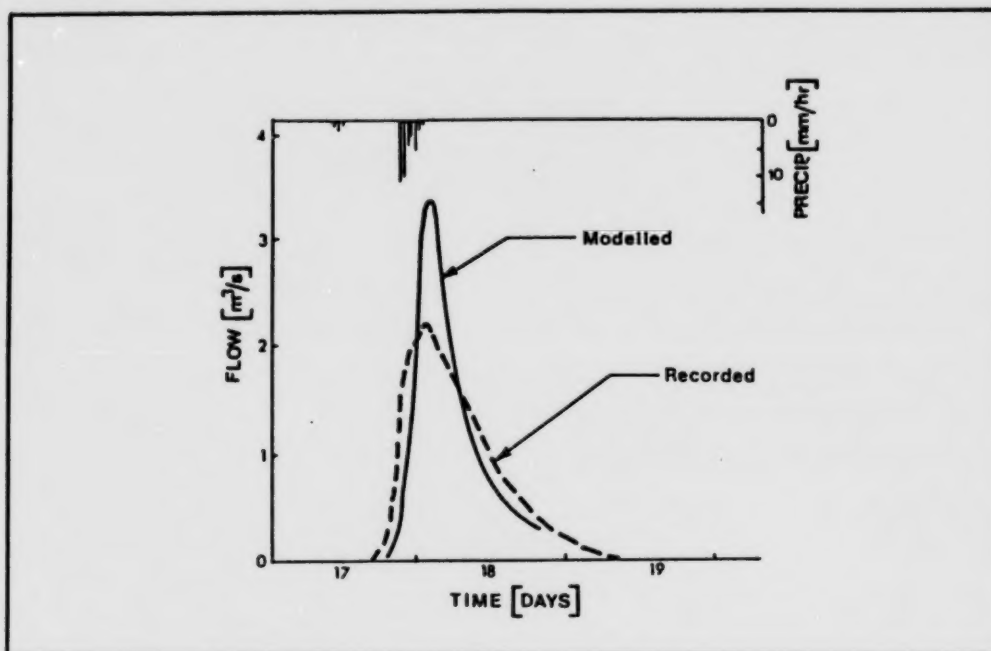
** Values less than those for the 12 hr. storm



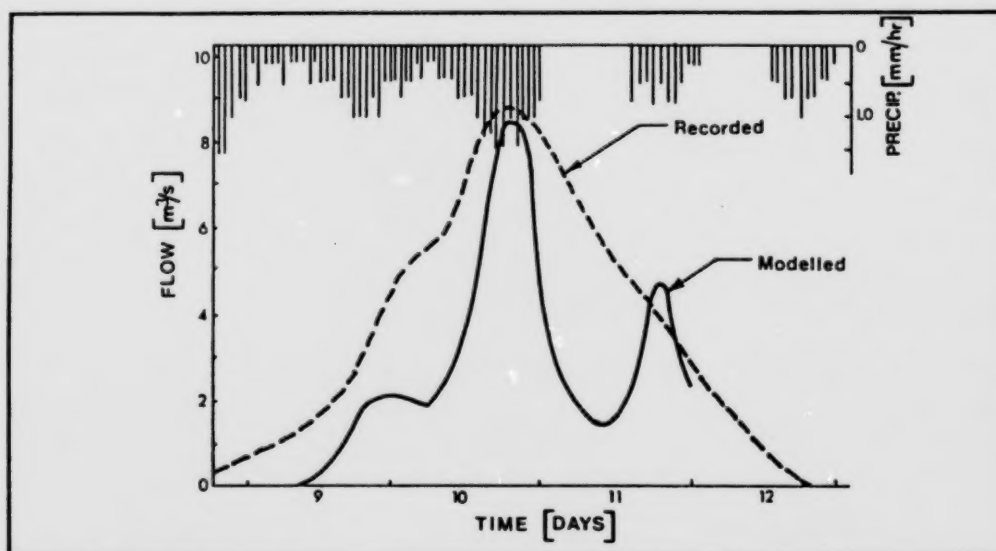
Spear MacDonald
& Associates Ltd

Cumming-Cockburn
& Associates Ltd

FIGURE 3.2



AUGUST, 1981 EVENT



APRIL, 1981 EVENT

CANADA - NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM

**COMPARISON OF
MEASURED AND SIMULATED
FLOWS**

WALKER BROOK AT W.S.C.
GAUGE

Spear, MacDonald
& Associates Ltd

Cumming - Cockburn
& Associates Ltd

FIGURE 3.3

4.0 HYDRAULIC ANALYSES

4.1 General

The main purpose of the hydraulic analyses was to transform peak discharge estimates into flood profiles along the study reach. This was undertaken by utilizing a mathematical model to simulate water surface profiles corresponding to the 1:20 and 1:100 year flood events. The effect of channel and floodplain storage on flood profiles along the study reach were not considered to be significant. In addition, it was postulated that tidal variations were not a significant factor in determining flood levels along the study reach (this was subsequently confirmed by undertaking model sensitivity testing; refer to section 4.4.4.) In such cases it is standard practice to assume steady state flow conditions in the computation of the backwater profiles.

Where a steady state backwater computation is employed, the appropriate design peak discharge input to the model is the instantaneous peak of the flood hydrograph.

In the case of gradually varied steady flow, the equations of continuity and momentum describing the one-dimensional flow can be simplified to the form of the well-known Bernoulli equation:

$$\frac{\partial h}{\partial x} = (S_0 - S_f) / (1 - v^2/gh) \quad (4.1)$$

where h = depth of flow (m)

x = distance in direction of flow (m)

S_0 = bottom slope

S_f = boundary frictional effect

v = velocity in direction of flow (m/s)

g = acceleration due to gravity (m/s²)

For natural channels, energy losses occur due to flow resistance. The resulting friction slope can be determined from the Manning's equation:

$$S_f = (nv/R^2/3)^2 \quad (4.2)$$

where n = Manning's roughness coefficient

R = hydraulic radius

In order to simulate the flood levels associated with each of the 1:20 and 1:100 year flood peaks, a mathematical model was developed to simulate the hydraulic characteristics of the Walker Brook watercourse. The model selected for application in the project is discussed in Section 4.2. The channel reach modelled consisted of that section of the Brook beginning at its outlet to the Restigouche River in the City of Campbellton and extending some 13 km upstream to a point near the intersection of Routes 270 and 275 south of the Community of Val D'Amour.

The computer model was developed to simulate the existing hydraulic characteristics of the watercourse based on the channel and floodplain configuration as interpreted from the existing topographic mapping and the results of comprehensive field topographic and reconnaissance surveys (see Section 4.3). Subsequent to model development and sensitivity testing (see Section 4.4), the backwater model was then utilized to establish the flood profiles associated with the peak 1:20 and 1:100 year discharge rates. The flood profiles are discussed in Section 4.7. The effects of spill from the main channel and the potential for ice jams are discussed in Sections 4.5 and 4.6 respectively.

4.2 Model Selection

The HEC-2 Model (USCE, 1979) was selected for use in simulating flood profiles along the study reach. This model has been successfully used in many similar practical applications. The program calculates water surface profiles for flow in natural or man-made channels, assuming that such flow is steady and gradually varied. The simplified one-dimensional equations of continuity and motion are solved using the standard step method with energy losses due to friction evaluated by the Manning's equation. The model can take into account the following factors:

1. Channel roughness
2. Floodplain roughness
3. Islands or flow divisions
4. Bends in the stream or floodplain
5. Cross-sectional area of the stream channel and floodplain
6. Slope of the channel and floodplain
7. Energy losses at hydraulic structures, including bridges, culverts, weirs, dams, etc.
8. Channel and floodplain expansion and contraction losses
9. Variation in discharge along the study reach (i.e. due to tributary inflows.)

The model requires input of channel and floodplain cross-sections and associated hydraulic parameters at frequent locations along the study reach. The cross-sections are normally located where changes occur in slope, cross-sectional area or channel roughness, and at bridges or culverts.

A major advantage of the HEC-2 model is that the channel and floodplain roughness (Manning's 'n') can be varied for each cross-section in the program. This allows a description of the various factors on which the roughness coefficient depends such as channel morphology, type and extent of vegetation, etc.

Energy losses created at hydraulic structures, such as bridges and culverts, are computed in the program in two parts. First the energy losses due to expansion and contraction of the flow at the cross-section on the upstream and downstream sides of the structure are calculated, and second, the energy loss through the structure itself is computed by either using the special bridge or the normal bridge sub-routine. Energy losses due to expansion and contraction of flow are calculated by employing expansion and contraction coefficients which are multiplied by the absolute difference in velocity heads between cross-sections to estimate the energy loss caused by the transition.

When the normal bridge sub-routine is used the water level is computed at the bridge section in the same manner as normal river cross-sections, but excluding the cross-sectional area of the bridge piers below the water surface. When the water surface elevation exceeds the bottom cord, the wetted perimeter of the

A total of 43 cross-sections were field surveyed along the study reach. The complete inventory of cross-sections, including location and extent is described in detail in the supplementary report on the physical surveys and field program. Cross-section measurements were obtained at representative locations along the study reach, and cross-sections were located based on changes in the slope, cross-sectional area or channel roughness. Additional cross-sections were measured near bridge and culvert crossings. Typical cross-sections for selected stream reaches are shown on Figure 4.1, and plots for each location are given in the supplementary report.

Each of the hydraulic structures along Walker Brook represents a potential flow constriction which may have a pronounced effect on water surface profiles during flood periods. Therefore, the physical dimensions and elevations of all hydraulic structures were field surveyed as described in detail in the supplementary report. These measurements included the size of the opening and the elevations of the soffit and deck of bridges and culverts. Floodplain and channel roughness coefficients were also assessed in the field. An inventory of photographs documenting the present conditions of hydraulic structures and floodplain and channel characteristics is also available as part of the supplementary report.

In order to collect peak water level data for the purpose of calibrating the hydraulic routing model, a total of eight crest gauges were installed along Walker Brook during the week of April 19, 1982. Subsequent measurements of peak discharge and water levels were undertaken in an attempt to collect data suitable for model calibration and verification. The locations of the gauging stations and data collected are discussed in detail in the supplementary report on the physical surveys and the field program. Unfortunately, during the period of operation of the field monitoring program, no significant discharge events, ice jam or debris floods occurred. As a result, no discharge or water level data suitable for calibration and verification of the hydrologic and hydraulic models was collected during this study.

The results of the field investigations and river and floodplain characteristics are discussed in the following section.

section is also adjusted. The special bridge routine computes losses through the structure for low flow or for any combination of weir flow and pressure flow.

The HEC-2 model was selected since it is a well-proven and well-documented non-proprietary technique which is flexible to use. The model can be applied in the future to evaluate the effects of recommended hydraulic improvements and proposed channelization along the study reach, etc. In addition, the model calculates the critical depth at each cross-section and can calculate profiles for supercritical flow, where required. This feature is especially useful for steep stream reaches such as those which occur along the Walker Brook study reach. Backwater profiles can be run for subcritical flow conditions by specifying a starting water level at the downstream end of the stream reach being simulated. For supercritical flow conditions flood profiles can be computed by starting the computation at a known water level at the upstream end of the study reach. For more detailed description of the HEC-2 program, reference is given to the HEC-2 water surface profiles user's manual, produced by the U.S. Army Corps of Engineers (USCE, 1979).

4.3 Development of Hydraulic Model

4.3.1 Field Survey

As indicated in Section 2.3, some cross-sectional data were available as a result of previous field surveys undertaken by Environment New Brunswick (King, 1978). However, only minimal use could be made of this information since recent changes to the channel and floodplain have occurred along the study reach. Therefore, field surveys were undertaken as part of the present study to measure typical channel and floodplain cross-sections.

All topographic information collected during the field surveys was related to geodetic elevation and all sections were located by means of traverse surveys related to the New Brunswick grid system. A significant effort was also made to properly document the results of the field surveys in a "Supplementary Report : Physical Surveys and Field Program" in order to ensure that the data collected could be utilized in future investigations.

4.3.2 Channel and Floodplain Characteristics

i) General Hydraulic Characteristics

In order to assess the hydraulic condition of the Walker Brook floodplain, a review of the channel and floodplain characteristics of the study reach was undertaken by means of a detailed examination of the study area during the field reconnaissance surveys, and through the interpretation of the available mapping and background information.

The Walker Brook channel can be characterized according to two main reaches. These being upstream and downstream of Highway 11 near Sugarloaf Provincial Park (refer to Table 4.1). Upstream of Sugarloaf, the channel is in a more natural state and is winding and fairly steep with an average slope of 0.018 m/m. The channel follows the course of Route 270 through Val D'Amour. The channel banks are relatively clean and well defined while the channel bed is comprised of stones and cobbles. There is little channel vegetation present due to the high flow velocities and steep bedslope associated with the watercourse, except for localized areas upstream of Val D'Amour and near the snowmobile crossing to Sugarloaf. The channel in this reach is relatively narrow (± 3 m) and fairly deep (2-3 m). A number of small access bridges have been constructed to cross the channel, particularly in the Val D'Amour area.

Downstream of Sugarloaf, the man-made influences on the Walker Brook channel are more diverse, with extensive areas of rip-rap protection. Some areas of bank erosion and sediment deposition in the lower portion of the watercourse are evident. Some channel and floodplain characteristics for the reach below Highway 11 are summarized in Table 4.1.

Similarly, the floodplain characteristics of Walker Brook also differ substantially upstream and downstream of the Sugarloaf-Highway 11 area. Upstream of Sugarloaf, the overbank areas are heavily vegetated with trees and brush in most areas. The width of the floodplain is relatively narrow due to the steep valley confining the watercourse. In general, the eastern overbank has a relatively higher potential flow capacity since the channel follows Route 270. However,

the western overbank is generally a mixture of forest and residential usage which would tend to constrict the passage of flow.

Through the City of Campbellton, the overbank areas are characterized by extensive residential and commercial development. The floodplain in this area is relatively wide and the damage potential to structures is high. Table 4.1 and Figure 4.2 illustrate the typical channel and floodplain characteristics of the study reach.

Manning's roughness coefficients were selected dependent on relative description and density of a particular type of land use and soil cover, type and amount of vegetation, channel configuration and natural physical constraints relative to the channel and overbank reaches along the watercourse. Typical roughness coefficients were determined based on field observations of channel and overbank characteristics and with reference to the classification derived by Chow (Chow, 1959). A summary of typical Manning's roughness coefficients determined for various reaches of the study area are given in Table 4.1. It should be noted that while the effect of man-made physical obstructions such as buildings, etc. are reflected in the determination of Manning's values, expansion and contraction coefficients were primarily used to account for flow constrictions due to structures.

Rapidly varying consecutive sections, bridge and culvert entrances and outlets, floodplain structures, etc. were accounted for through adjustments in the expansion and contraction coefficients, following recommended values found in the HEC-2 manual (USCE, 1979). The general criteria for determining these coefficients are summarized in Table 4.2, together with typical values.

ii) Hydraulic Structures

The discharge and flood levels during peak flows are also controlled in part by some 32 bridges and culverts and four weirs, from its outlet to the upstream study limit. Many of these structures create a severe constriction to the watercourse and are frequently surcharged, thus increasing the flood risk to residences in the adjacent floodplain. However, a number of these structures along the upstream reaches are private stream crossings which could easily be

washed out during the occurrence of peak flow events. Table 4.3 provides a summary of the bridge and culvert characteristics for each structure along Walker Brook. The culvert capacities summarized in Table 4.3 were estimated assuming that the road elevation is not exceeded. The following briefly describes the hydraulic characteristics of the most potentially serious constrictions on the watercourse.

1. Village Road (B4):

As historical records indicate, this structure is perhaps the most susceptible to surcharging of any along the watercourse. Low culvert conveyance capacity combined with a high potential for ice and debris jamming at this location have resulted in numerous flooding problems in the vicinity of this structure. Accumulation of sediment and debris during flood events have also blocked the inlet at this structure, thus creating a severe backwater effect resulting in overtopping of Village Road.

2. Gallant Drive (B5):

The existing road and bridge deck elevation at this location is low, and combined with the small opening of the Gallant Drive bridge has resulted in severe surcharging during many high flow events in the past. A large amount of sedimentation has reduced the flow conveyance capacity of this structure in recent years. Due to the overbank configuration near Gallant Drive, the backwater created by the bridge and road crossing creates a broad floodplain with depths of up to one metre determined for severe flood events. A high potential for ice and debris jamming at this location also increases the severity of the flood risk in this area, as historical records have indicated.

3. Dover Road (B7):

The overbank in this reach is characterized as being broad and relatively flat. The limited discharge capacity of the culvert combined with the flow constriction caused by the road crossing creates a backwater in the channel upstream of this structure. Flood waters back up until surcharging the road, thus increasing the flood hazard to the structures in the adjacent low lying overbank areas.

4. Highway 11 Overpass (B11):

The Highway 11 road culvert capacity is limited due to spill in the roadside ditch in an easterly direction along Highway 11. The ditch capacity is exceeded during high flow events at a point near the Gallant Road crossing. Spill over the highway back to Walker Brook occurs at this location. In addition, some spill occurs in a westerly direction around the Route 270 interchange. This spill aggravates downstream flooding in the area of the municipal garage and the shopping mall. Historical records have shown one to two metre flood depths on Route 270 in this area and 0.5 metre depths over Highway 11 near Gallant Road. Additional discussion on the spill analyses is presented in Section 4.5.

5. Church Bridge to Graveyard (Val D'Amour) (B26):

A limited channel capacity at this location combines with the low conveyance capacity of the Church bridge to create a severe backwater condition along Walker Brook. Records on historical flooding along this reach also confirm that this area is highly susceptible to flooding.

Surcharging at other culverts and road crossings also occurs to a lesser extent at other locations, as identified in Section 4.7. Additional details on bridges and culverts, including field sketches made at each location are available in the supplementary report.

4.3.3 Model Structure

In order to simulate the flood levels associated with the 1:20 and 1:100 year peaks, the available background and field data was input to the HEC-2 program. Figure 4.2 shows in schematic form the HEC-2 model structure developed for the hydraulic analysis.

With respect to input of available data, the following criteria were established in order to define cross-section locations and characteristics.

- i) All sections are coded as if looking upstream along the watercourse.

- ii) Field measured cross-sections as applied in the hydraulic model are correlated to the supplementary field report by maintaining the same cross-section numbers, substituting the alphabetic character "S" with the numeral "5". For example, field surveyed cross-section S-42 is identified as section number 542 in the backwater model.
- iii) In some cases, field measured cross-sections were used more than once as typical cross-sections along particular reaches. Elevations were adjusted by applying the average slope between measured sections. Repeated sections are denoted in the backwater model by a number to the right of a decimal point following the original section number identification (e.g. 42.1).
- iv) All hydraulic structures are referenced to the field survey report through the use of comment cards. For ease in identification, numerals to the right of a decimal point equalling "8" represent bridges and culverts, while "88" represents dams or weirs.

The layout of cross-sections is shown on Figure 4.2 which illustrates in schematic form the application, extent and relative location of the field measured sections as input to the HEC-2 model.

With respect to the modelling of the hydraulic structures on the watercourse, all structures were simulated using the special bridge method, as documented in the HEC-2 Users Manual (USCE, 1979), with the exception of the following:

- i) Municipal Water Main Truss (B3)
This structure was modelled as a constricted area section as the bridge supports and deck has no constrictive effect on flood flows
- ii) Gerrard Street Footbridge (B6)
This structure was modelled as a constricted area section. The bridge deck would not hold up under a peak flow event and would not control backwater conditions upstream. However, the reduced conveyance area of the channel at this location would affect flood levels in the area.

iii) Sugarloaf Park Bridge (B12)

This structure is effectively perched, therefore the normal bridge method was applied as documented in the HEC-2 Users Manual since discharge would occur over the approaches to the bridge.

iv) Ruins of Private Bridge at Body Shop (B32)

Modelled as a constricted section as the bridge deck was washed out during the 1976 flood on Walker Brook.

v) The fourth culvert crossing on Route 270, just downstream of the study limit. At the time of the field survey this culvert was fully blocked by the debris. As no potential damage would result to this area, and no effect on flood levels upstream would result from backwater at this location, this crossing was deemed hydraulically insignificant and was thus not modelled.

As stated previously, the special bridge method was applied to all other structures. Thus, the most accurate method for calculating hydraulic losses through each structure was utilized.

Preliminary HEC-2 simulations indicated that the flow regime was supercritical along parts of the study reach. Therefore, backwater simulations were undertaken assuming both subcritical and supercritical flow conditions. That is, backwater models were developed for Walker Brook beginning at the upstream and downstream sections for supercritical and subcritical runs respectively. Based on a detailed review of the subcritical and supercritical profiles, a combination of both models was selected to most accurately represent flood levels along the watercourse.

This was done by following procedures which are outlined in the HEC-2 modelling documentation. For a subcritical profile computation, if a channel reach consisting of three or more consecutive sections passes through critical depth, then the supercritical flow profile should be applied. Therefore, given the above condition, a combination of both backwater model simulations was utilized to determine the Walker Brook flood profiles.

It should be noted that both supercritical and subcritical profiles yielded comparable results in terms of water levels. The maximum differences found at any section was in the order of 0.7 m between subcritical and supercritical elevations, while the average difference was in the order of 0.2 m.

Boundary conditions and other model input parameters were determined as follows:

- i) All computations were initially performed assuming a condition of free flow. That is, all hydraulic structures were considered to be free from any temporary obstructions which would reduce their effective discharge capacity during the passing of peak storm flows. The model was subsequently revised during the assessment of ice and debris jam potential to account for blockage at various culverts on the system. (As discussed in detail in Section 4.6 of this report, ice and debris jams and their effect on water levels and flood damages are more important for frequent flood events.)
- ii) The hydraulic coefficients used in the analysis were derived as previously outlined in Section 4.3.2.
- iii) The available data base was inadequate to calibrate the backwater model. Therefore, some sensitivity analyses were undertaken as outlined in Section 4.4. Various parameters and assumptions applied in the backwater modelling were tested in order to assess their relative influence on the computed flood levels along the watercourse.
- iv) Preliminary simulations indicated that significant spill from the channel occurs at several locations. The model was, therefore, rerun with the flow lost from the channel removed from the relevant downstream sections of the reach, as discussed in Section 4.5.

The final results of the flood profile computations are discussed in Section 4.7.

4.4 Sensitivity Testing

4.4.1 General

In order to assess variations in the magnitude of the design flows on flood profiles along the Walker Brook study reach, various sensitivity simulations were undertaken.

Based on a review of the initial model simulations, the field reconnaissance survey, and on previous results of backwater modelling on similar watercourses, the following parameters were found to be of primary importance with respect to definition of flood levels in the study area:

- peak discharge rates along the watercourse
- definition of channel and floodplain roughness coefficients (Manning's 'n')
- the influence of tidal variations on the control of outflow from Walker Brook to the Restigouche River
- expansion and contraction coefficients

The following sections outlined the methodology applied, and results obtained, from the sensitivity testing of hydraulic model parameters.

4.4.2 Sensitivity to Computed Peak Discharge

Sensitivity simulations were conducted utilizing the computed 1:100 year peak discharge versus the 1:100 year peak discharge at the upper and lower 95% confidence limits. The 1:20 year flows were not varied since the 1:100 year lower 95% confidence limit peak flow was very similar to the median 1:20 year estimate (see Table 4.4). Table 4.4 summarizes the range of discharge as applied in this analysis as determined from the Regional Flood Frequency Analysis.

As expected, the peak discharge had some effect on variation of water levels, as indicated in Table 4.5, with the average difference being 0.38 m, above the mean at the +95% confidence level. The smaller differences were found

to be associated with the steeper stream reaches while the largest differences were found to be in the downstream flood plain. Similarly, at the -95% confidence level, an average decrease of 0.33 m in water levels compared to those computed for the mean.

The water surface profiles with $\pm 95\%$ confidence bands are summarized in Table 4.5. In summary, it was found that the average difference of the median profile from the $\pm 95\%$ ("upper and lower profiles") was found to be 0.35 m along the study reach.

Comparing these analyses to sensitivity tests discussed in subsequent sections, it was evident that changes in flow had the most effect on water levels computed for the entire length of Walker Brook. However, the large variations in discharge resulted in relatively smaller variations in water level profiles along the study reach.

4.4.3 Sensitivity to Roughness Coefficient

Manning's roughness coefficients for the channel and floodplain were determined as described in Section 4.3.2. The sensitivity of the flood profile computations to variations in roughness coefficient was also undertaken as a means of further substantiating the accuracy of the backwater model.

The range of Manning's "n" values applied in the analysis are summarized in Table 4.6. The discharge value used in the sensitivity testing corresponded to the median 1:100 year estimates of peak flow.

The range of n-values given in Table 4.6 corresponds to the range of potential values as described by Chow (Chow, 1959) applied to the channel characteristics of the study reach. For the purposes of these sensitivity tests, it was determined that the roughness coefficients could vary $\pm 20\%$ about the "mean value" previously discussed. Table 4.7 presents a comparison of computed water levels for variations in roughness coefficients. The average difference in water levels about the mean were found to be 0.06 m and 0.05 m corresponding to +20% and -20% changes in the roughness coefficients respectively.

This corresponds to an average difference between the upper and lower range of about 0.06 m along the study reach.

On the basis of these tests, it was concluded that there is little appreciable variation in peak flood levels along the watercourse as a result of possible variations in roughness coefficients. The sensitivity analysis verifies the accuracy of the selected coefficients for application in the backwater modelling of the 1:20 and 1:100 year design flood events in the absence of suitable data for model calibration and validation.

4.4.4 Sensitivity to Tidal Influence

The HEC-2 model requires the definition of starting water levels to initiate the backwater computations. Sensitivity simulations were undertaken to determine the effect of variations in the initial water levels on flood levels in the lower portion of Walker Brook which may result from the tidal influence from the Restigouche River. The tidal characteristics used in the analysis were previously discussed in Section 2.3 (see Table 2.1).

For the purpose of this investigation, the range of water levels at the Walker Brook outlet was chosen as 1.1 m to 1.9 m. This represents a range from the mean high tide to the maximum high tide at the outlet of the Brook. As little tidal information is available for Campbellton, the values chosen were derived by transferring recorded tide levels available for the Pointe St. Pierre station (Government of Canada, 1982).

Table 4.8 shows the results of the sensitivity analysis on the influence of tidal levels at the outlet of the watercourse on flood profiles at upstream locations. Tides were found to have no effect on flood levels upstream of the old dam which is located just downstream from Village Road.

The water surface profile below the remains of the old dam is the only area subject to tidal influence during flood events. This reach is characterized by a broad channel with high banks and, therefore, variations in starting water levels does not have a significant influence on the extent of the floodplain or on flood hazard characteristics in this reach.

4.4.5 Sensitivity to Expansion and Contraction Coefficients

Sensitivity simulations were also undertaken in order to assess the effect of variations in expansion and contraction coefficients on the accuracy of the backwater model. Simulations were conducted by varying the expansion and contraction coefficients about a range of $\pm 50\%$ from the typical mean values given in Table 4.2. From this analysis, it was found that negligible differences in water levels resulted (variations less than ± 0.1 m resulting about the mean). It was, therefore, concluded that flood profiles along the Walker Brook watercourse are insensitive to variations in expansion/contraction coefficients. The mean values given in Table 4.2 were, therefore, applied for the purpose of this investigation.

4.4.6 Summary of Results and Conclusions

The following points summarize the main findings and conclusions of the sensitivity analyses on computed flood profiles along the Walker Brook watercourse:

- 1) The flood profiles along the watercourse were relatively more sensitive to variations in peak discharge, as represented by the following confidence limits:
 - average difference above the mean for +95% confidence limit was 0.38 m
 - average difference below the mean for -95% confidence limit was 0.33 m
- 3) The sensitivity analysis on the flood profiles along the watercourse to variation in roughness coefficient resulted in minimal changes in flood elevation (average of 0.06 m over $\pm 20\%$ change in 'n' values). The flood profiles along the watercourse are, therefore, not sensitive to variations in roughness coefficient due to the hydraulic regime and steep bed slope.

- 3) The influence of tidal variations are felt only in the extreme lower portion of the watercourse; specifically, from the Restigouche River up to the remains of the old dam which is located downstream of Village Road.
- 4) The flood profiles were not sensitive to variations in the expansion and contraction coefficients.

4.5 Spill Analysis

4.5.1 Spill Areas

Analysis of initial backwater simulations revealed that during peak flow events, a significant amount of spill from the channel would occur at the following locations:

1. Old dam downstream from Sugarloaf Park
2. Upstream of the culvert on Hwy. 11
3. Val D'Amour area

The location of the first two spill areas are depicted on Figure 4.3. The third spill area is located at cross-section S-9 in Val D'Amour.

The loss of flow from the channel and floodplain could affect downstream flood profiles both upstream and downstream of the spill location. Rating curves were developed for each spill section using field survey information, backwater levels in the channel and Manning's equation for the spill section. The rating curves for each spill section are given on Figure 4.4.

The following briefly describes the hydraulic condition at each of the spill areas.

1. Old Dam Downstream from Sugarloaf Provincial Park

This structure is in very poor condition and is no longer operated as a flow control dam. However, a significant amount of sedimentation has occurred on the upstream side of the structure resulting in a significant drop in the

bottom profile at this location (drop is about 3.1 m). Downstream of the structure, the existing channel can accommodate the 1:100 year flow. However, the flow constriction caused by the remains of the dam, the sedimentation upstream, combined with the man-made alterations to the natural floodplain (specifically the access ramp to the Highway 11 overpass) creates a spill condition in the area.

It was found that the ramp acts as a flow divide which directs the overbank discharge along Route 270 towards the City of Campbellton rather than draining back towards the channel downstream. At high discharge rates, the spilled flow would, therefore, bypass the stream reach where the Water Survey of Canada Hydrometric Station is located, passing instead under the Highway 11 overpass and eventually rejoining the channel in the vicinity of the municipal garage. (for the purposes of this study, it was assumed that the spilled flow rejoins the channel at cross-section S-36.)

In order to estimate the rate of spill during peak flow conditions, a rating curve was developed for the spill section just upstream of the dam (see Figure 4.3), as previously discussed. The results of this analysis indicated that 10.8 m³/s and 11.1 m³/s would be lost from the channel and floodplain during the occurrence of the 1:20 and 1:100 year flood events respectively. This spill represents a loss of about 20% and 15% respectively for the 1:20 and 1:100 year peaks. The backwater profiles were then recomputed with the reduced flows applied between the old dam and the municipal garage. Downstream of section S-35 this spilled flow was added back to the total flow and included in the flood profile calculated along downstream reaches.

2. Upstream of the Culvert on Highway 11

The local topographic characteristics just upstream of this culvert, combined with the relatively low invert in the ditch running northeasterly along Highway 11, results in a substantial spill from the channel during peak flow events. The rating curve determined for the Highway 11 culvert is shown on Figure 4.3. The invert of the ditch is some 1.5 m below the obvert of the culvert.

The ditch acts as an overflow channel during flood events, with discharge re-joining Walker Brook via a culvert under Highway 11 just upstream of the Gallant Drive overpass. During large floods, Highway 11 is also overtopped in this area before this excess flow reaches Walker Brook. It is estimated that the culvert crossing under Highway 11 just upstream of Gallant Drive only has capacity to pass about 15 m³/s. Therefore, discharge in excess of this amount along the ditch would overtop Highway 11.

In order to estimate the amount of spillage to the ditch, a HEC-2 backwater model was developed for the ditch from the culvert near Gallant Drive up to the Highway 11 culvert. A rating curve for the ditch control section was then developed by inputting a range of discharge values to the backwater model. The rating curve is shown on Figure 4.3. The Walker Brook backwater model was then used in conjunction with the rating curve developed for the ditch in order to determine spill amounts.

Based on this analysis it was determined that 22 m³/s and 40 m³/s would be lost from the channel and floodplain of Walker Brook for the 1:20 and 1:100 year peak flows respectively. This represents a significant loss of some 38% and 48% of the 1:20 and 1:100 year peak flows respectively.

The backwater model along Walker Brook was then revised with the reduced peak flows applied between Highway 11 and the stream location just upstream of Gallant Drive.

3. Val D'Amour Area

In general, it was found that the main channel of Walker Brook can accommodate discharge rates up to the 1:100 year peak flow in the Val D'Amour area. However, numerous hydraulic structures constrict the available capacity at some locations. As a result of backwater from these structures, a localized flow diversion from the main channel occurs between sections S-9 and S-11. A small amount of flow would leave the channel and follow Route 270 in this area. The spill is localized and rejoins the main channel just downstream. Additional detailed topographic mapping of this area would be required to accurately define the extent and nature of the relatively small spill volumes from the channel in this area.

Due to the small amount of spill and localized nature, it was assumed that for the purposes of this investigation, no adjustments to the computed water surface profiles were required in this area.

4.5.2 Summary and Conclusions

1. The spill analyses confirmed that a significant loss of flow from the channel and floodplain would occur at two locations, namely:
 - i) Old Dam upstream from Sugarloaf Park
 - 1:20 year spill $10.8 \text{ m}^3/\text{s}$
 - 1:100 year spill $11.1 \text{ m}^3/\text{s}$
 - ii) Upstream of Culvert on Highway 11
 - 1:20 year spill $22 \text{ m}^3/\text{s}$
 - 1:100 year spill $40 \text{ m}^3/\text{s}$
2. Spill in the Val D'Amour area was found to be localized and was assumed to be insignificant for the purposes of this investigation.
3. The design flow rates for input to the Walker Brook hydraulic model were therefore, adjusted in order to account for spill amounts. The revised flow rates as applied in the backwater model are summarized in Table 4.9.
4. The Water Survey of Canada gauging station should be moved or alternatively the old dam upstream should be removed to minimize spill. Otherwise it may not be possible to extend the rating curve in the future.

4.6 Ice Jam Potential

4.6.1 General

The state-of-the-art in describing the formation and effects of ice jams continues to evolve through an active research program being carried on across

Canada. The occurrence of ice jams may occur during all phases of ice cover formation, growth and decay. However, on the Walker Brook watershed, it is the frequent and significant accumulation of ice during the spring break-up that has resulted in historical flooding problems at various locations.

The discharge rate and hydraulic characteristics of the river channel significantly affect the potential for ice jam formation. Ice jams are generally associated with a combination of river discharge and the strength of the ice during the freshet period. The major factor which initiates destruction of the solid ice cover is the hydromechanical uplift exerted by increasing discharge. This can be caused by light rains, increased temperatures, wind and snowmelt. As the flow in Walker Brook increases, the cover fractures at various points and ice floes begin moving downstream. The break-up rate and transport of ice is, therefore, generally associated with peak flows during the spring freshet.

In a situation where there is little or no rainfall during the melt period and very gradual snowmelt, the river discharge will not increase rapidly. In this case, strong hydromechanical uplift forces are not developed and the river ice cover might deteriorate in place under the influence of thermal factors.

Ice jams generally form at constricted sections and locations where irregularities occur. Some characteristic locations where ice jams could initiate include:

1. At a transition zone between a rapidly flowing stream reach and a section of more tranquil flow. On Walker Brook this occurs at the confluence with the Restigouche River;
2. At channel singularities such as shoals, changes in alignment, constrictions in the flow and other channel obstructions;
3. At locations of hydraulic structures such as bridges, weirs, and dams along the stream;
4. At locations where significant accumulations of anchor ice have formed along the stream.

The following sections discuss the potential for the occurrence of ice jams and their effects along Walker Brook.

4.6.2 Historical Ice Jam Locations

Ice and debris jams have frequently been observed to occur along Walker Brook at the following locations:

1. In the interval area at the site of the old dam ruins just downstream of Village Road
2. On the upstream side of Village Road
3. On the upstream side of Gallant Drive
4. At a bend and constriction in the channel just upstream of Gerard St.

Discussions with local residents together with a review of available background information (Folster, 1980) indicated that the remains of the old dam causes a flow obstruction at which ice and debris frequently jams. However, flood damages associated with jams at this location have apparently been small. This is attributed to the lack of development in the adjacent floodplain at this location.

Ice and debris jams at the Village Road and Gallant Drive crossings occur frequently during peak flow conditions. This is apparently due to the severe constriction in flow section at these locations. Debris is frequently removed at these sites by the Campbellton Works Department.

A small debris jam was observed at Village Road during October 1982. This location has frequently had water over the road as a result of ice and debris jams completely plugging the culverts.

According to information obtained from local residents, ice and debris frequently accumulate in the Brook at a bend and constriction in the stream just upstream of Gerard Street. The City of Campbellton has apparently given some previous consideration to realignment and widening of the channel at this location.

No other detailed information describing the characteristics of ice and debris jams is currently available for Walker Brook.

4.6.3 Ice Characteristics

Very little information on ice jam formation, ice thickness and floe sizes are available for Walker Brook. However, based on preliminary field surveys carried out by Environment New Brunswick and the Water Survey of Canada (see Section 2.3) during 1981 and 1982, it is apparent that development of a continuous ice cover on Walker Brook below the Water Survey of Canada gauge likely occurs each year. On the other hand, it is apparent that only a partial ice cover generally forms over those reaches of the brook upstream from the Water Survey of Canada gauge. This is likely due to the high flow velocities combined with a significant insulation of the Brook due to the large depth of accumulated snow each winter.

The available observations have also indicated that the depth of flow in the channel during the winter is very shallow and that the stream commonly freezes to the bottom in some locations. Due to the insulation by snow and the relatively shallow depths of flow, it appears that the maximum thickness of ice cover formed during the winter season would not exceed .5 metre with more common ice thicknesses being in the range of .25 to .40 metres. Assuming an average thickness of .40 metres and assuming 100% cover below the Water Survey gauge and only 50% cover above the Water Survey gauge, the volume of ice at break-up in these reaches would be approximately 15,000 m³ and 5,000 m³ respectively.

It is assumed that this volume of ice might be available for ice jam formation during the spring freshet. Due to velocity and stream channel characteristics, it is apparent that the largest volume of ice is formed over the lower portion of the study reach. This results in a much smaller potential for ice jam formation in upstream areas.

No information was available on diameter of ice floes during the spring freshet, however, floe sizes of 1-2 m diameter are likely typical along Walker Brook. It should be pointed out that formation of anchor ice is suspected at some locations; however, at the present time no specific information is available.

4.6.4 Ice Jam Factors

The available information on ice and ice jam characteristics are sparse along Walker Brook. Using state-of-the-art techniques, the following factors can be used to describe ice formation and ice jams along Walker Brook:

1. Ice thickness
2. Geometry of ice floes
3. Froude number of the flow
4. Velocity of the flow
5. Depth of the flow

The potential for ice jam formation was analysed by using existing theoretical relationships together with computed hydraulic characteristics provided by analysing the HEC-2 backwater model output.

The flow velocity and depth values calculated from the HEC-2 model were utilized to estimate Froude numbers and velocity indices in order to assess the possibility of ice jam formation. Typical hydraulic indices were computed for several representative reaches of the study area and are summarized in Table 4.10. The hydraulic parameters were based on the 1:20 year flood peak as it was assumed that this discharge rate would be more typical of maximum flow conditions during the spring freshet.

The possibility of jam formation was examined relative to the flow velocity in Walker Brook. Generally speaking it could be assumed that ice floes could become submerged thus increasing the potential for jam formation and/or blockage of culverts or bridge openings if the flow velocity in the stream exceeds about 0.75 metres per second.

A typical Froude number (Kivisild, 1959) was also calculated for each of the study reaches shown in Table 4.10. On this basis it was concluded that since the Froude number exceeded the critical value of 0.08 to 0.13, then a potential for ice jam formation exists all along Walker Brook. On the other hand, the flow velocities and discharge rates along Walker Brook are likely such that in

the absence of man-made constrictions, ice floes would likely be swept downstream and ice jams would not likely form at most channel locations. This finding is consistent with the formation of ice and debris jams mainly at man-made constrictions in the channel.

In summary, it was found that flow velocities along Walker Brook are much higher than 0.75 m/s. As a result, ice floes moving downstream could easily become submerged at the leading edge of an ice jam, or at other obstructions to flow such as at culverts and bridges, etc. This could result in the plugging of undersized bridges or culverts. The high velocities would also have a similar effect on debris in the channel and explains why the undersized culverts and bridges are easily blocked by ice or debris during peak flow conditions. For example, our analyses have indicated that complete blockage of the culverts at Village Road likely occurred during the 1973 flood.

The plugging of culverts and bridges significantly affects flow capacity and in many cases results in water discharging over the road at the problem locations.

The effect on water levels was analysed by assuming complete blockage of the road crossings at the locations shown in Table 4.11. The results are also summarized in Table 4.11 which indicates the flood level in relation to the road elevation at each location for various flow rates. (Road elevations are the lowest elevation along the overflow section.) The findings of this analysis indicated that a very small increase in the depth of flow over the road can accommodate large increases in discharge, assuming the culverts are completely blocked. This means that even for relatively small peak discharge events, if the culverts are blocked, then the resulting water level is similar in magnitude to the design level open water flood profiles. This is due to the fact that these openings are presently undersized resulting in overtopping of the roads during open water conditions. This finding is important in regard to future estimates of flood damages associated with frequent flood events.

Reference to Figure 4.5 (found at the end of this chapter) and Table 4.11 also indicates that the blockage of these culverts results in a relatively localized backwater due to the relatively steep stream profile.

4.6.5 Summary of Ice and Debris Jam Potential

The following points summarize the results of the analysis of the potential for ice jam formation:

1. Since the largest volume of ice is apparently produced each year below the Water Survey of Canada gauge, a larger potential for ice jams exists in the lower portion of the study area.
2. The review of available background information has indicated that significant ice and debris jams have occurred mainly at those locations along the channel where man-made constrictions to the flow are present. Specifically these are:
 - a) the dam ruins (minor problems)
 - b) Village Road
 - c) Gallant Drive
 - d) Channel bend and constriction just upstream of Gerard Street.
3. Ice and debris jams at Village Road and Gallant Drive tend to have a significant effect on flow conveyance at these locations. Blockage of these and other culverts at low discharge rates would result in water levels roughly equivalent to 1:20 to 1:100 year open water levels. Blockage at high discharge rates results in minor increases in water levels since the culverts are undersized at these locations and discharge would occur over the road (i.e. the capacity of weir flow over the road is much larger than the culvert capacities).
4. The potential for damage producing ice jams can be reduced by eliminating significant man-made constrictions to the flow regime. The culverts at Village Road and the bridge opening at Gallant Drive should be enlarged. The channel upstream of Gerard Street should be realigned and widened.
5. Additional information on ice jam formation, ice thicknesses and flow characteristics during the spring freshet need to be collected during future monitoring programs.

4.7 1:20 and 1:100 Year Flood Profiles

The main objective of this investigation was to determine flood profiles along the study reach for floods with a recurrence interval of 1:20 and 1:100 years.

The hydrologic analyses described in Chapter 3.0 resulted in the determination of 1:20 and 1:100 year instantaneous peak discharge values for the study area (see Table 3.2). The sensitivity testing described in Section 4.4 confirmed the importance of accurate discharge estimates. A subsequent spill analysis described in Section 4.5 resulted in some adjustments to the peak discharge values, as summarized in Table 4.9.

The HEC-2 backwater model was developed as discussed in Section 4.3 and a schematic of the overall model structure is given on Figure 4.2. Channel and floodplain characteristics were determined as discussed in Section 4.3.2. For example, typical channel and floodplain roughness values are summarized in Tables 4.1 and 4.6. The sensitivity testing of the hydraulic model has indicated that in the absence of suitable calibration and validation data, the hydraulic parameters presently determined for Walker Brook result in an adequately accurate representation of the hydraulic characteristics for the purpose of this investigation.

The numerical values of the resulting flood profiles are summarized in Table 4.12 for both discharge conditions (i.e. adjusted for spill and unadjusted). The "unadjusted" flood profile would be more representative of conditions where the old dam near Sugarloaf Mountain is removed and assuming that the ditch along Highway 11 is blocked or for some other reason is not operating as an overflow system. The "spill adjusted" flood profile is representative of existing conditions assuming spill occurs at these locations as previously described. However, at the present time, it is believed that blockage due to ice or debris could occur at the spill locations. Therefore, because of the possibility of blockage of spill at these locations, it is recommended that the assessment of flooding conditions in downstream areas should be based on the "unadjusted" flood profiles.

The flood profiles corresponding to these backwater simulations are also plotted on Figures 4.5 and 4.6 (as found at the end of this chapter) for the "unadjusted" and "spill adjusted" discharge conditions respectively.

The extent of the flooded areas along the lower portion of Walker Brook were plotted on existing topographic maps at a scale of 1:1200. The index of available maps is given as Figure 4.7. The available mapping extends to just upstream of Dover Street near the Campbellton City Limits. Upstream of Campbellton there is presently no suitable topographic mapping for determining the extent of the flooded areas.

From the confluence of the Brook with the Restigouche River up to Village Road, the flood hazard is low since the 1:20 and 1:100 year "unadjusted" flood lines do not encroach on any existing urban developments.

Upstream of Village Road to Dover Road, the flood hazard is higher since overtopping of both Village Road and Gallant Drive occurs for both the 1:20 and 1:100 year events. Few buildings are directly affected by flooding in this reach, however, access could be restricted to Lily Lake Road and the Provincial Hospital.

A moderate flood hazard exists in the reach from Dover Road to Pine Street. The "spill adjusted" flow is confined in the channel and floodplain, while the 1:100 year "unadjusted" flow would inundate some structures in the vicinity of Pine and 11a Streets.

From Pine Street up to the extent of available mapping, the flood hazard is very high. The extent of the "unadjusted" 1:100 year flood line would inundate many structures (± 160). On the other hand, if the ditch along Highway 11 acts as an overflow, then the flood hazard is significantly reduced (± 40 structures). For the case of adjusted flows, much of the channel from Pine Street to Dover Road could contain the 1:20 year adjusted flow, with only a few structures affected by flooding.

The general and widespread flooding of the area upstream of Dover Road would be fairly shallow with typical depths of about 0.8 m and 0.3 m for the "unadjusted" and "spill adjusted" flood levels respectively.

The flooding of developed areas as described above relates to overland flow in the floodplain areas. In addition, some basements adjacent to the floodplain may also be flooded due to infiltration. However, the determination of this type of basement flooding was beyond the scope of the present study.

Upstream of Highway 11, the peak flows are generally contained in the natural channel and floodplain with the exception of those spill areas discussed in Section 4.5. The average top width of flooding (in those areas where spill does not occur) was found to be about 16 m, compared to an average channel top width of 3 m. The main flooding hazards along the upstream reaches would appear to be related to high discharge velocities (2 to 4 m/s) and to the washout of wooden access bridges at various locations along the study area.

Detailed topographic maps are required upstream of Highway 11 in order to identify the exact extent of flood prone lands. Also, the first-floor elevations and structure openings of all potentially flood prone structures should be field surveyed in order to determine flood damages in the next phase of the investigations.

4.8 Main Conclusions and Recommendations of Hydraulic Analyses

The HEC-2 backwater model was successfully utilized to calculate flood profiles along the study reach using channel and floodplain characteristics determined from the field surveys.

The main conclusions and recommendations of the hydraulic analyses are summarized as follows:

Conclusions

1. Supercritical flow profiles were encountered at various locations along the study reach. However, the difference between subcritical and

supercritical flow simulations was found to be relatively insignificant along these reaches (differences less than 0.2 m on average).

2. The parameters sensitivity analysis for flood profiles along the watercourse revealed that flood levels were relatively more sensitive to variations in peak discharge compared to variations in Manning's 'n', tidal influence or expansion and contraction coefficients. The average difference above and below the mean flows for the $\pm 95\%$ confidence limits was found to be 0.38 and 0.33 m respectively.
3. The effect of tides was evident only in the extreme lower portion of the watercourse. Upstream of the old dam location which is located downstream of Village Road the tidal variations had no effect on flood levels.
4. The spill analyses revealed that significant spill occurs at two locations, namely:
 1. The old dam downstream from Sugarloaf Park
 2. The overpass of the Highway 11 bridge at Route 270.The design flow rates for the Walker Brook hydraulic model were, therefore, adjusted in order to account for spill amounts.

Recommendations

1. The potential for significant ice jams along Walker Brook exists mainly at man-made flow restrictions. This includes the stream crossing at Village Road and the bridge at Gallant Drive. These structures must be enlarged to reduce the potential for future ice and debris jam formation.
2. Additional hydraulic information should be collected for the purpose of future model calibration and verification. This includes data on ice jam formation and ice thickness, together with information on discharge and water levels along the study reach.

3. The 1:20 year and 1:100 year flood profiles for Walker Brook are summarized on Figures 4.5 and 4.6 (unadjusted and adjusted spill respectively) and the water levels are given in a tabular comparison in Table 4.12. The unadjusted flood profiles should be used in assessing flood conditions along the study reach.
4. Additional detailed topographic mapping along the study reach should be obtained in order to more accurately identify the extent of flooding.

TABLE 4.1
Typical Channel and Floodplain Characteristics
along Walker Brook

Reach	Channel Depth (m)	Typical Channel Characteristics	Channel Slope (m/m)	Channel Width (m)	Typical Floodplain Characteristics Vegetation/Conditions	No. of Bridges & Structures	Typical Manning's "n" Left Overbank	Typical Manning's "n" Right Overbank
Val D'Amour to Highway 11	2-3	3 - stone & cobble - some grass	0.018	30	- dense to moderate vegetation	25	0.035	0.040
Hwy. 11 to Alexander St.	3-4	10 - cobble & stone	0.016	40	- grass land with some pavement - clean floodplain	2	0.035	0.030
Alexander St. to Gerrard St.	2-3	15 - cobble & stone	0.0093	55	- grassland & pavement	3	0.035	0.035
Gerrard Street to Village Road	1-2	20 - cobble	0.0034	105	- moderate vegetation - scrub & grassland	2	0.040	0.035
Village Road to Outlet	0.5-1	50 - stone & boulders	0.0021	65	- scrub grass - some debris - some brush	4	0.040	0.040

TABLE 4.2
Summary of Typical Expansion
and Contraction Coefficients (1)

Parameter	Typical Values
Expansion Coefficient:	
i) Gradually varying sections	0.5
ii) Rapidly varying sections and hydraulic constraints	0.8
Contraction Coefficients:	
i) Gradually varying sections	0.3
ii) Rapidly varying sections and hydraulic constraints	0.5

(1) Source (USCE, 1979)

TABLE 4.3

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Bridge and Culvert Characteristics

Location	Type of Structure	Elevations		Area of Opening (m ²)	Approximate Capacity (m ³ /s)
		Road (m)	Soffit (m)		
B1 - Ramsey St.	Bridge			59.8	f(tide)
B2 - CNR Line	"	8.2	5.5	N/A	f(tide)
B3 - Water Main	"	5.2	3.4	N/A	f(tide)
D1 - Dover & Matheson	Dam	1.6	N/A	N/A	N/A
B4 - Village Road	3 CMP Culverts	6.3	3.5	8.3	44
B5 - Gallant Dr.	Conc. Box Culvert	7.7	7.1	7.7	22
B6 - Gerrard St.	Wood Bridge	12.5	12.2	20.9	100
B7 - Dover Rd.	Conc. Encased CMP culvert	16.2	15.6	11.0	44
B8 - Ila & Pine	" " "	22.1	21.6	9.8	40
B9 - Alexander St.	Wood Bridge	24.4	24.3	15.0	49
B10 - Dover Rd. (NBEPC)	CMP Culvert	30.9	29.7	9.2	45
B11 - Hwy. 11	" "	34.5	33.2	10.7	67
D2 - Old Dam by WSC Gauge	Dam Ruins	38.1	N/A	N/A	N/A
B12 - Sugarloaf	Bridge	42.9	41.2	25.0	100
D3 - Sugarloaf Weirs	Weirs	40.5	N/A	N/A	N/A
		41.4			
B13 - Snowmobile	Twin CMP Culverts	54.5	51.8	11.1	100
B14 - Private Dr.	Wood Bridge	54.8	54.1	12.2	60
B15 - Sta. A	Masonry Bridge	69.5	69.2	9.0	26
B16 - Private Drive	Wood Bridge	75.1	74.8	13.6	50
B17 - Gulch Rd.	" "	93.7	92.9	10.6	42
B18 - Gulch Rd. Dam	Conc. Dam	94.2	N/A	N/A	N/A
B19 - Gulch Rd. Walkway	Wood Bridge	100.6	100.1	15.0	30
B20 - Private Road	Wood Bridge	100.4	100.1	7.6	24
B21 - Rte 270 & Malauze Rd.	Wood Bridge	106.2	105.8	12.4	25
B22 - Private Road	" "	108.7	108.3	7.2	20
B23 - Private Road	" "	108.8	108.5	6.1	17
B24 - Gravel Pit Rd.	CMP Culvert	111.3	110.5	5.7	20
B25 - Lily Lake Rd.	" "	114.5	114.0	6.9	20
B26 - Church Br.	Wood Bridge	115.9	115.5	12.2	7
B27 - Rte 270 - 1st Crossing	CMP Culvert	117.5	117.1	4.8	15
B28 - Private Road	Wood Bridge	117.6	117.2	6.0	12
B29 - Rte. 270 - 2nd Crossing	CMP Culvert	129.1	126.5	4.8	13
B30 - Private Road	Wood Bridge	133.9	132.7	7.5	720
B31 - Private Road	" "	133.4	133.0	5.5	720
B32 - Bridge Ruins	" " Abut.	135.3	N/A	N/A	N/A
B33 - Rte. 270 - 3rd Crossing	CMP Culvert	136.3	134.6	3.4	12

Note: B1 : denotes bridge number

D1 : denotes dam (or weir) number

CMP : corrugated metal pipe

N/A : not applicable

NBEPC : New Brunswick Electric Power Commission

f(tide) : function of tidal elevation

TABLE 4.4
Maximum Instantaneous Discharge Applied
in Sensitivity Analysis

Section No.	Location	Q_{100} (+95% C.L.) (m ³ /s)	Q_{100} (m ³ /s)	Q_{100} (-95% C.L.) (m ³ /s)	Q_{20} (m ³ /s)
43.21	Outlet	82.9	51.8	32.6	57.8
527	WSC Gauge	76.4	47.8	30.1	53.4
525	Sugarloaf Park	61.1	38.2	24.0	42.4
517	Gulch Rd. (confluence with Cold Brook)	52.0	32.5	20.5	36.2
514	Conf. with Lily Lake Brook	29.1	18.2	11.4	20.3
13.81	Val D'Amour	22.2	13.9	8.7	15.5
504	Near Study Limit	14.3	8.9	5.6	10.0

Note: 1) Discharge based on Regional Flood Frequency analysis (Section 3.2.2)
2) Sensitivity simulations assume no adjustment due to spill analyses.

TABLE 4.5
Sensitivity of Water Levels to
Variations in Discharge

Section No.	Location	1:100 Year Water Level in Metres		
		-95% Conf. Limit Q100	Mean Q100	+95% Conf. Limit Q100
43.21		1.10	1.10	1.10
543	Ramsey Street	1.15	1.27	1.65
542		1.20	1.39	1.86
541	Municipal Water Main	1.30	1.58	2.13
540		1.32	1.60	2.15
539		1.38	1.67	2.19
538		2.84	3.12	3.49
537.8	Village Rd.	3.53	3.71	3.99
536.8		7.16	7.24	7.51
535.8	Gerrard St.	9.20	9.51	9.97
534		12.34	12.86	13.13
533		16.12	16.72	16.91
532		18.18	18.48	18.83
531.8	Ila & Pine	20.65	21.20	21.48
530.8	Alexander St.	23.67	24.10	24.66
528	d/s Hwy. 11	30.33	30.41	30.64
527	W.S.C. Guage	35.92	36.42	37.07
526		38.61	38.74	38.94
525	d/s Sugarloaf Park	41.49	41.63	41.87
524	Sugarloaf Weirs	41.86	42.16	42.47
523		41.88	42.19	42.51
521	d/s Snowmobile Crossing	50.25	50.81	51.41
520		52.51	52.74	54.09
519		62.34	62.48	62.60
518		71.83	72.02	72.25
517		89.67	89.98	90.40
516.88	Gulch Rd. Dam	92.91	93.79	94.27
515		100.63	101.30	101.67
514		102.43	102.66	102.98
513	Malauze Rd.	105.35	105.74	106.33
512		108.63	109.04	109.69
510	U/S Lily Lake Rd.	114.21	114.55	115.23
511		115.70	116.00	116.29
509		122.87	123.15	123.67
508		128.77	129.69	129.70
507		135.41	136.51	137.03
506		147.93	148.08	148.25
505		169.23	169.33	169.45
504		186.68	186.83	187.01
503		193.54	193.77	194.15
502		208.75	208.95	209.20
501	U/S Study Limit	263.12	263.18	263.25

Note: Only surveyed cross-sections denoted at selected locations

543 denotes 5-43

516.88 denotes Gulch Rd. Dam

531.8 denotes Ila and Pine Bridge

TABLE 4.6
Summary of Roughness Coefficients

Section No.	Location	Mannings Roughness Coefficient							
		Left Overbank		Channel		Right Overbank			
		-20%	+20%	-20%	Mean	-20%	Mean	-20%	+20%
43.21	Outlet	0.025	0.035	0.035	0.048	0.025	0.035	0.035	0.045
542		0.025	0.035	0.030	0.040	0.025	0.035	0.035	0.045
42.83		0.025	0.035	0.035	0.048	0.025	0.035	0.035	0.045
539	d/s Village Rd.	0.035	0.045	0.025	0.035	0.050	0.060	0.060	0.070
537.8		0.030	0.040	0.025	0.035	0.030	0.040	0.040	0.050
534		0.020	0.030	0.020	0.030	0.020	0.030	0.030	0.040
33.5	d/s of Dover Rd.	0.025	0.035	0.030	0.040	0.025	0.035	0.035	0.045
30.1		0.025	0.035	0.020	0.030	0.025	0.035	0.035	0.045
28		0.025	0.035	0.020	0.030	0.025	0.035	0.035	0.045
25.8	d/s of Hwy 11	0.020	0.030	0.030	0.040	0.020	0.030	0.030	0.045
21.1		0.030	0.040	0.035	0.045	0.030	0.040	0.040	0.050
20.1		0.020	0.030	0.030	0.040	0.040	0.050	0.050	0.060
16.2	v/s Snowmobile Crossing	0.030	0.040	0.025	0.035	0.060	0.070	0.070	0.080
14.1		0.025	0.035	0.025	0.035	0.050	0.060	0.060	0.070
9.83		0.025	0.035	0.030	0.040	0.025	0.035	0.035	0.045
509	d/s Malauze Rd.	0.025	0.035	0.035	0.045	0.025	0.035	0.035	0.045
8.86		0.025	0.035	0.030	0.040	0.025	0.035	0.035	0.045
505		0.025	0.035	0.030	0.040	0.025	0.035	0.035	0.045
502	Bridge ruins at Body Shop	0.040	0.050	0.030	0.040	0.060	0.080	0.080	0.100
		0.030	0.040	0.030	0.040	0.050	0.060	0.060	0.070
		0.060	0.070	0.030	0.040	0.060	0.070	0.070	0.080

NOTE: 542 denotes section S-42
537.8 denotes Village Rd. Bridge

TABLE 4.7
Sensitivity of Water Levels to
Variation in Roughness Coefficient

Section No.	Location	1:100 Year Water Level in Metres		
		-20% "n"	Mean "n"	+20% "n"
43.21		1.10	1.10	1.10
543	Ramsey Street	1.24	1.27	1.29
542		1.35	1.39	1.47
541	Municipal Water Main	1.49	1.58	1.61
540		1.52	1.60	1.63
539		1.64	1.67	1.72
538		3.06	3.12	3.24
537.8	Village Rd.	3.54	3.71	3.96
536.8		7.16	7.24	7.34
535.8	Gerrard St.	9.51	9.51	9.53
534		12.86	12.86	12.86
533		16.69	16.72	16.74
532		18.37	18.48	18.58
531.8	Ila & Pine	20.67	21.20	21.42
530.8	Alexander St.	24.10	24.10	24.35
528	d/s Hwy 11	30.40	30.41	30.43
527	W.S.C. Guage	36.24	36.42	36.61
526		38.74	38.74	38.75
525	d/s Sugarloaf Park	41.64	41.63	41.70
524	Sugarloaf Weirs	42.12	42.16	42.20
523		42.15	42.19	42.24
521	d/s Snowmobile Crossing	50.81	50.81	50.83
520		52.56	52.74	52.84
519		62.49	62.48	62.46
518		72.02	72.02	72.02
517		89.99	89.98	89.98
516.88	Gulch Rd. Dam	93.79	93.79	93.79
515		101.30	101.30	101.30
514		102.66	102.66	102.74
513	Malauze Rd.	105.58	105.74	105.87
512		108.88	109.04	109.15
510	U/S Lily Lake Rd.	114.46	114.55	114.76
511		115.78	116.00	116.10
509		123.15	123.15	123.15
			9.69	129.18
506		148.08	148	
505		169.33	169.33	169.36
504		186.84	186.83	186.86
503		193.77	193.77	193.76
502		208.95	208.95	208.97
501	U/S Study Limit	263.18	263.18	263.18

Note: Only surveyed cross-sections denoted at selected locations
 543 denotes S-43 (identified in Supplementary Report : Physical Surveys
 and Field Program
 516.88 denotes Gulch Road Dam
 531.8 denotes Ila and Pine Bridge
 For Q100 "mean discharge" by Regional Flood Frequency Analysis

TABLE 4.8
Comparison of Computed 1:100 Year Water Levels
Through Variation in Tidal Level

<u>Section No.</u>	<u>Location</u>	<u>Mean High Water Level (1.1 m)</u>	<u>Maximum High Water Level (1.9 m)</u>
43.21	Outlet	1.10	1.9
543.8	Ramsey Street	1.20	1.92
543		1.27	1.94
542		1.39	1.99
42.8	CNR Main Line Bridge	1.50	2.04
42.81		1.50	2.04
42.82		1.50	2.04
42.83		1.50	2.04
541	Municipal Water Main	1.58	2.09
540		1.60	2.10
539		1.67	2.12
539.88	Old Dam near Dover & Matheson	1.67	2.12
39		2.82	2.85
538		3.12	3.14
537.8	Village Rd.	3.71	3.71
37.0	U/S Village Rd.	6.61	6.61

NOTE: 543 denotes section S-43

42.8 denotes bridge

539.88 denotes dam

For Q₁₀₀ mean discharge by Regional Flood Frequency Analysis

TABLE 4.9
Summary of Peak Flows and Revised Peak Flows
to Account for Spill

Section No.	Location	Maximum Instantaneous Discharge*			
		1:20 Year		1:100 Year	
		<u>Unadjusted</u>	<u>Adjusted</u>	<u>Unadjusted</u>	<u>Adjusted</u>
43.21	Outlet	57.8*	57.8	82.9*	82.9
534	d/s of Dover Rd.	57.8	35.8*	82.9	42.9*
528	d/s Hwy. 11	57.8	35.8*	82.9	31.8*
27	d/s Hwy. 11	57.8	25.0*	82.9	71.8*
527	WSC guage	53.4	42.6*	76.4	65.3*
526	u/s of old dam	53.4	53.4*	76.4*	76.4
525	Sugarloaf Park	42.4	42.4	61.1*	61.1
517	Cold Brook Confluence	36.2*	36.2	52.0*	52.0
514	Lily Lake Confluence	20.3*	20.3	29.1*	29.1
13.81	Val D'Amour	15.5*	15.5	22.2*	22.2
504	Near 4th Crossing	10.0*	10.0	14.3*	14.3

* Applied in Backwater Modelling

TABLE 4.10
Hydraulic Characteristics
and Ice Jam Potential

<u>Location of Flow</u>	<u>Typical Velocity (m/s)</u>	<u>Critical Velocity (m/s)</u>	<u>Typical Flow Depth (m)</u>	<u>Typical Froude Number</u>
Val D'Amour to Campbellton	3.0	0.61	1.13	0.189
Hwy 11 to Alexander	1.95	0.81	2.00	0.183
Alexander to Gerrard	1.95	0.75	1.71	0.182
Gerrard to Village Road	1.01	0.72	1.6	0.182
Village Road to confluence	1.10	0.88	2.38	0.182

- NOTES: 1. Based on 1:20 year discharge values
2. If typical velocity is greater than critical velocity there may be a high potential for ice jam formation.
3. If Froude number B is greater than 0.08 to 0.13 there may be a high potential for the ice jam formation.

TABLE 4.11
Water Levels with Ice and Debris Jams

Location	Condition of Culvert/Bridge	Road Elevation (m)	Water Level at Indicated Location (d) (m) Discharge Rate (m ³ /s)			
			(a) 15.0	(a) 26.6	(b) 57.8	(c) 82.9
Village Rd. (B4)	Plugged	6.3	6.72	6.84	7.06	7.20
	Open (e)		3.83	4.64	6.73	6.94
Gallant Dr. (B5)	Plugged	7.7	8.16	8.28	8.52	8.66
	Open (e)		7.23	7.90	8.30	8.49
Dover Rd. (B7)	Plugged	16.2	14.21	15.02	17.01	17.04
	Open (e)		14.21	15.02	17.0	17.04
Alexander St. (B9)	Plugged	24.5	24.51	24.53	24.95	25.01
	Open (e)		23.14	23.48	24.67	24.88
Highway 11 (B11)	Plugged	34.5	34.51	34.53	34.85	34.91
	Open (e)		32.34	33.31	33.34	34.82

(a) Assumed values representative of more frequent spring flood events

(b) 1:20 year flood peaks (m³/s)

(c) 1:100 year flood peaks (m³/s)

(d) Water level just upstream of culvert (m)

(e) Open water level for comparison purposes (m)

Table 4.12

Summary of Computed Water Surface Elevations

SECTION NO.	LOCATION/ DESCRIPTION	1:20 YEAR WATER LEVEL IN FEET		1:100 YEAR WATER LEVEL IN FEET	
		UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL	UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL
43.210	Outlet to Restigouche R.	1.10	1.10	1.10	1.10
543.800	Ramsey St.	1.21	1.23	1.52	1.52
543.000		1.32	1.32	1.65	1.65
542.000		1.45	1.45	1.86	1.86
42.800	C.N.R. Main Line	1.57	1.57	2.01	2.01
42.810		1.57	1.57	2.01	2.01
42.820		1.57	1.57	2.01	2.01
42.830		1.58	1.58	2.01	2.01
541.000	Truss for Water Main	1.66	1.66	2.13	2.13
540.000		1.68	1.68	2.15	2.15
539.000		1.75	1.75	2.19	2.19
539.800	Old Dam near Dover & Matheson	1.75	1.75	2.19	2.19
39.000		2.91	2.91	3.32	3.32
538.000		3.19	3.19	3.49	3.49
537.800	Village Rd.	3.76	3.76	3.93	3.93
37.000		6.68	6.68	6.92	6.92
37.100		6.69	6.69	6.93	6.93
536.800	Gallant Drive	7.30	7.30	7.50	7.50
36.000		8.30	8.30	8.59	8.59
35.100		8.36	8.36	8.55	8.55
36.200		8.45	8.45	8.65	8.65
36.300		8.76	8.76	8.94	8.94
535.000		9.50	9.50	9.95	9.95
35.100		9.50	9.47	9.93	9.93
35.200		9.59	9.88	9.96	9.96
35.800	Gerrard Foot Bridge	10.28	10.28	10.36	10.36
35.400		10.58	10.68	10.78	10.78
35.500		11.11	11.11	11.18	11.18
35.600		11.39	11.39	11.42	11.42
534.000		12.20	12.20	12.47	12.47
34.100		13.25	12.33	13.45	12.60
34.800		15.42	14.63	16.76	14.96
34.810		16.99	15.57	16.99	15.57
33.100	Dover St.	17.03	16.30	17.06	16.61
33.200		17.03	16.30	17.06	16.61
533.000		17.04	16.33	17.07	16.62
33.500		17.07	16.44	17.11	16.66
32.100		17.34	17.03	17.56	17.16
32.150		17.73	17.46	18.03	17.54
32.200		18.01	17.73	18.25	17.83
532.000		18.56	18.24	18.83	18.35
32.300		18.70	18.38	18.93	18.50
32.400		18.92	18.61	19.14	18.75
32.500		19.21	18.92	19.44	19.02
31.100		20.30	19.78	21.13	19.96
531.800	11a & Pine Street	21.34	20.76	21.48	20.97
31.200		22.40	21.88	22.51	22.10
30.100		22.41	21.97	22.53	22.61
30.200		22.83	22.41	23.54	22.55
30.250		23.60	23.13	24.07	23.29
530.300		24.21	23.75	24.89	23.91
	Alexander St.				

TABLE 1.12
Summary of Computed Water Surface Elevations

SECTION NO.	LOCATION/ DESCRIPTION	1:20 YEAR WATER LEVEL IN METRES		1:100 YEAR WATER LEVEL IN METRES	
		UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL	UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL
30.300		24.69	23.71	24.88	24.26
30.400		24.65	24.29	25.15	24.40
30.500		24.95	24.55	25.67	24.68
30.600		25.43	25.01	26.16	25.15
30.700		25.73	25.30	26.45	25.45
29.100		26.64	26.01	26.73	26.11
29.200		26.65	26.15	26.75	26.20
29.300		27.01	26.58	27.17	26.54
29.400		27.50	27.26	27.63	27.35
29.800	Dover Rd. Near N.B.E.P.C.	29.45	29.28	29.48	29.30
29.540		29.55	29.52	29.70	29.52
28.000		29.10	29.62	29.95	29.67
528.000		30.24	29.79	30.64	29.79
28.800	Highway 11	33.18	31.81	34.63	33.15
28.100		33.22	33.07	34.79	33.77
27.000		35.15	33.39	35.16	33.15
27.100		35.16	33.50	35.16	33.25
27.200		35.19	33.73	35.20	33.48
27.300		35.26	34.16	35.27	33.71
27.400		35.40	34.74	35.47	33.90
27.500		35.68	35.34	35.97	33.77
527.000	Water Survey of Canada Gauge	36.55	36.31	37.07	36.76
26.880		37.52	37.32	37.93	37.75
526.600	Old Dam Below Sugarloaf	38.65	38.62	38.67	38.67
26.000		38.76	38.71	38.94	38.76
26.100		38.81	38.75	38.96	38.70
26.200		39.03	39.02	39.18	38.91
26.300		39.55	39.55	39.72	39.72
26.400		40.03	40.02	40.16	40.16
26.500		40.53	40.53	40.75	40.75
26.550		41.05	41.03	41.15	41.15
525.000		41.67	41.67	41.67	41.67
25.800	Sugarloaf Park Bridge	42.09	42.09	42.20	42.20
25.100		42.10	42.10	42.22	42.22
25.200		42.12	42.12	42.26	42.26
25.300		42.13	42.13	42.27	42.27
24.000		42.23	42.23	42.47	42.47
24.880	Sugarloaf Weirs (1st)	42.24	42.24	42.47	42.47
24.100		42.26	42.26	42.50	42.50
24.200		42.26	42.26	42.50	42.50
523.000		42.27	42.27	42.51	42.51
23.880	Sugarloaf Weirs (2nd)	42.27	42.27	42.51	42.51
23.100		42.22	42.22	42.46	42.46
23.200		42.60	42.60	42.80	42.80
23.300		42.60	42.60	42.81	42.81
23.400		42.62	42.62	42.83	42.83
23.500		42.70	42.70	42.97	42.97
23.600		42.99	42.99	43.31	43.31
21.100		45.90	45.90	46.00	46.00
21.150		45.58	45.58	45.97	45.97
21.200		46.50	46.50	46.53	46.53

TABLE 4.12

Summary of Proposed Water Surface Elevations

SECTION NO.	LOCATION/ DESCRIPTION	1:20 YEAR WATER LEVEL IN METRES		1:100 YEAR WATER LEVEL IN METRES	
		UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL	UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL
21.300		46.49	46.49	46.85	46.85
21.400		47.09	47.09	47.39	47.39
21.500		47.80	47.80	47.79	47.79
21.600		47.78	47.78	48.19	48.19
21.700		48.70	48.70	48.84	48.84
21.710		49.39	49.39	49.62	49.62
21.720		49.79	49.79	50.01	50.01
21.730		50.29	50.29	50.51	50.51
21.800		51.19	51.19	51.41	51.41
21.800	Snowmobile Crossing	51.36	51.36	51.57	51.57
21.810		51.47	51.47	53.91	53.91
20.100		51.93	51.93	54.08	54.08
20.200		51.94	51.94	54.08	54.08
20.300		51.96	51.96	54.08	54.08
20.400		52.05	52.05	54.08	54.08
20.500		52.31	52.31	54.09	54.09
20.000		52.70	52.70	54.09	54.09
20.800	Private Drive u/s Sugarloaf	53.51	53.51	54.30	54.30
20.810		53.64	53.64	55.03	55.03
20.900		54.94	54.94	55.10	55.10
519.000		62.49	62.49	62.61	62.61
19.100		63.01	63.01	63.13	63.13
19.200		63.50	63.50	63.61	63.61
19.300		63.99	63.99	64.13	64.13
19.400		64.50	64.50	64.61	64.61
19.500		65.01	65.01	65.12	65.12
19.600		65.50	65.50	65.62	65.62
19.700		65.99	65.99	66.12	66.12
19.710		66.50	66.50	66.62	66.62
19.720		67.00	67.00	67.14	67.14
19.730		67.50	67.50	67.62	67.62
519.800	Private Bridge at Sta. A	69.26	69.26	69.26	69.26
19.810		69.80	69.80	69.94	69.94
518.000		72.06	72.06	72.25	72.25
18.800	Private Bridge u/s Sta. A.	74.20	74.20	74.88	74.88
18.810		74.91	74.91	75.45	75.45
517.000		90.07	90.07	90.40	90.40
17.800	Gulch Rd. Bridge	92.47	92.47	93.42	93.42
517.810		93.18	93.18	94.08	94.08
16.100		94.04	94.04	94.27	94.27
516.880	Gulch Rd. Dam	94.04	94.04	94.27	94.27
16.900		96.49	96.49	96.95	96.95
516.200		96.75	96.75	97.21	97.21
16.300		96.78	96.78	97.23	97.23
16.400		96.86	96.86	97.30	97.30
16.500		97.14	97.14	97.46	97.46
16.600		97.64	97.64	97.95	97.95
16.700		98.14	98.14	98.45	98.45

TABLE 4.12
Summary of Computed Water Surface Elevations

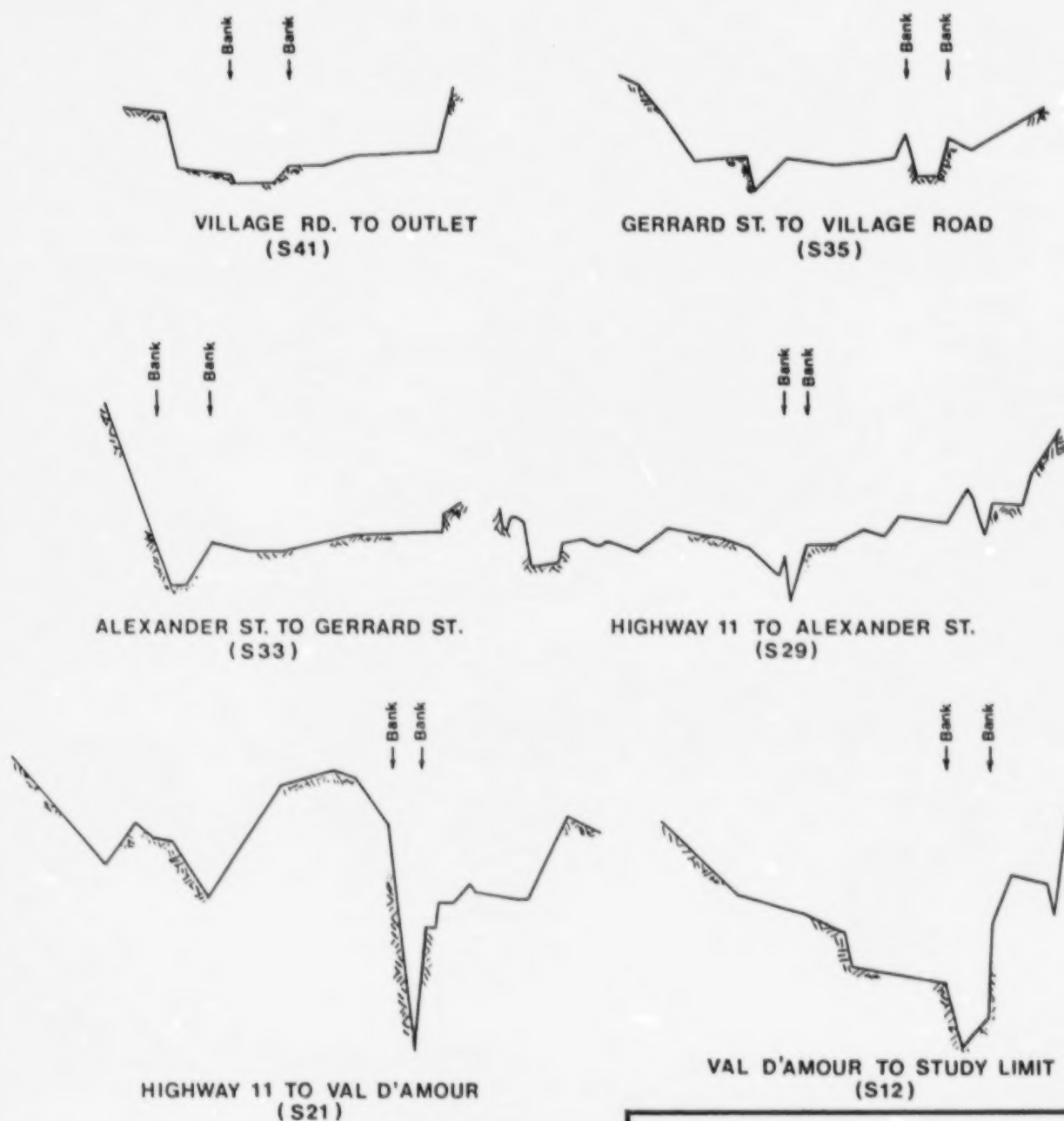
SECTION NO.	LOCATION/ DESCRIPTION	1:20 YEAR WATER LEVEL IN METRES		1:100 YEAR WATER LEVEL IN METRES	
		UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL	UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL
16.710		98.61	98.64	98.95	98.95
15.100		99.87	99.87	100.43	100.43
15.810		100.61	100.61	100.57	100.57
15.200		100.85	100.85	100.98	100.98
15.300		100.98	100.98	101.31	101.31
15.820		100.96	100.96	101.29	101.29
15.400	Walkway u/s Gulch Rd.	101.00	101.00	101.33	101.33
15.000		101.39	101.39	101.67	101.67
15.900		101.44	101.44	101.72	101.72
514.000		102.62	102.62	102.84	102.84
14.100		103.17	103.17	103.38	103.38
14.200		103.68	103.68	103.87	103.87
14.300		104.15	104.15	104.38	104.38
14.400		104.73	104.73	104.93	104.93
14.800	Malauze Rd. & Route 270	105.47	105.47	105.81	105.81
14.810		105.59	105.59	105.89	105.89
513.000	Val D'Amour	105.85	105.85	106.24	106.24
13.100		105.89	105.89	106.26	106.26
13.200		106.17	106.17	106.38	106.38
13.300		106.73	106.73	106.87	106.87
13.400		107.23	107.23	107.35	107.35
13.810	Private Bridge u/s Malauze Rd.	107.52	107.52	108.04	108.04
13.820		108.04	108.04	108.05	108.05
12.100		108.43	108.43	108.85	108.85
12.810	Private Bridge d/s Gravel Pit	108.51	108.51	108.90	108.90
12.820		108.67	108.67	109.13	109.13
512.000		109.11	109.11	109.65	109.65
12.840	Gravel Pit Road	109.83	109.83	110.23	110.23
12.850		109.84	109.84	110.25	110.25
10.100		110.32	110.32	110.44	110.44
10.200		110.49	110.49	110.60	110.60
10.300		110.62	110.62	111.01	111.01
10.400		111.30	111.30	111.35	111.35
10.500		111.39	111.39	111.62	111.62
10.600		111.73	111.73	111.95	111.95
10.700		112.13	112.13	112.35	112.35
10.710		112.08	112.08	112.23	112.23
10.800	Lily Lake Road	113.08	113.08	113.63	113.63
10.810		113.01	113.01	113.63	113.63
510.000		114.65	114.65	115.11	115.11
10.750		115.07	115.07	115.37	115.37
511.000		116.06	116.06	116.29	116.29
9.810	Church Bridge	116.34	116.34	116.62	116.62
9.820		116.37	116.37	116.67	116.67
9.100		116.44	116.44	116.75	116.75
9.830	CMP under Rte. 270 (1st)	116.60	116.60	117.61	117.61
9.840		116.60	116.60	117.90	117.90
9.200		116.52	116.52	117.98	117.98
9.850	Private Drive u/s Church	116.59	116.59	117.96	117.96
9.860		116.87	116.87	118.04	118.04
9.300		117.39	117.39	118.15	118.15
9.310		117.96	117.96	118.67	118.67
9.320		118.44	118.44	119.17	119.17
9.330		118.95	118.95	119.66	119.66
9.340		119.44	119.44	120.12	120.12

TABLE 4.12
Summary of Computed Water Surface Elevations

SECTION NO.	LOCATION/ DESCRIPTION	1:20 YEAR WATER LEVEL IN METRES		1:100 YEAR WATER LEVEL IN METRES	
		UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL	UNADJUSTED FOR SPILL	ADJUSTED FOR SPILL
9.350		120.16	120.16	120.67	120.67
9.360		120.66	120.66	121.17	121.17
9.370		121.16	121.16	121.66	121.66
9.380		121.66	121.66	122.15	122.15
9.390		122.16	122.16	122.60	122.60
509.000		123.24	123.24	123.65	123.65
9.010		123.95	123.95	124.16	124.16
9.020		124.34	124.34	124.65	124.65
9.030		124.84	124.84	125.16	125.16
9.040		125.34	125.34	125.64	125.64
9.050		125.85	125.85	126.16	126.16
9.060		126.34	126.34	126.63	126.63
9.070		126.84	126.84	127.10	127.10
9.090		126.89	126.89	127.67	127.67
8.800	CMP Under Rte. 270 (2nd)	128.16	128.16	129.12	129.12
8.810		128.17	128.17	129.42	129.42
508.000		129.56	129.56	129.54	129.54
8.010		129.55	129.55	129.51	129.51
8.020		129.46	129.40	129.68	129.68
8.030		129.40	129.40	130.15	130.15
8.040		129.91	129.91	130.47	130.47
8.050		130.39	130.39	130.54	130.54
8.060		130.66	130.86	131.01	131.01
8.820	Private Road	131.84	131.84	132.17	132.17
8.830		132.35	132.35	132.75	132.75
8.100		132.81	132.81	133.42	133.42
8.840		132.96	132.96	133.51	133.51
8.850	Private Bridge to Body Shop	132.97	132.97	133.66	133.66
8.200		133.51	133.51	133.97	133.97
8.860		133.97	133.97	134.28	134.28
8.870	Ruins of Bridge at Body Shop	134.35	134.35	134.74	134.74
7.100		134.86	134.86	135.36	135.36
7.800		135.42	135.42	135.86	135.86
7.810	CMP Under Rte. 270 (3rd)	136.62	136.62	136.97	136.97
507.000		136.67	136.67	137.03	137.03
506.000		148.11	148.11	148.25	148.25
505.000		169.36	169.36	169.45	169.45
504.000		186.88	186.88	187.01	187.01
503.000		193.81	193.81	194.15	194.15
502.000		209.00	209.00	209.20	209.20
2.100		239.00	239.00	239.21	239.21
501.000	Study Limit Near Rte. 270 & Rte. 275 Intersection	263.20	263.20	263.25	263.25

NOTE: For index to Section Nos. use criteria as follows:

- e.g. i) Field surveyed cross-section : SB denoted as 508.000
 ii) Duplicated and adjusted cross-section : Based on SB denoted as 0.010
 iii) Bridge : With overbank based on and adjusted by SB denoted as 11.000
 iv) Dam : With overbank based on and adjusted by SB denoted as 16.880



NOTE: TYPICAL SECTIONS LOOKING DOWNSTREAM.

CANADA - NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM

TYPICAL CROSS-SECTIONS

LEGEND

(S33) TYPICAL SECTION NUMBER

← BANK CHANNEL BANK LOCATION

Spear, MacDonald
& Associates Ltd

Cumming-Cockburn
& Associates Ltd

FIGURE 4.1

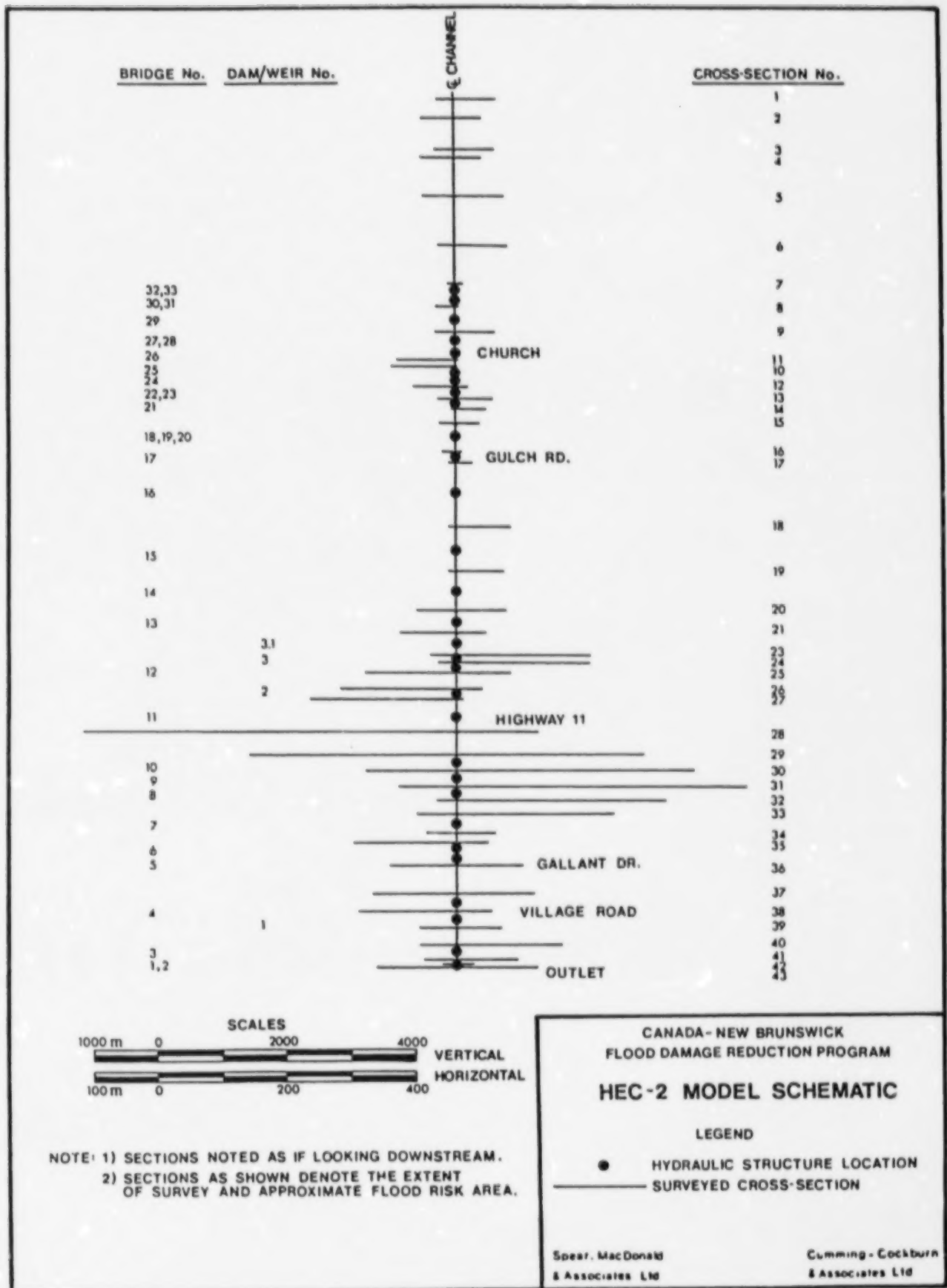
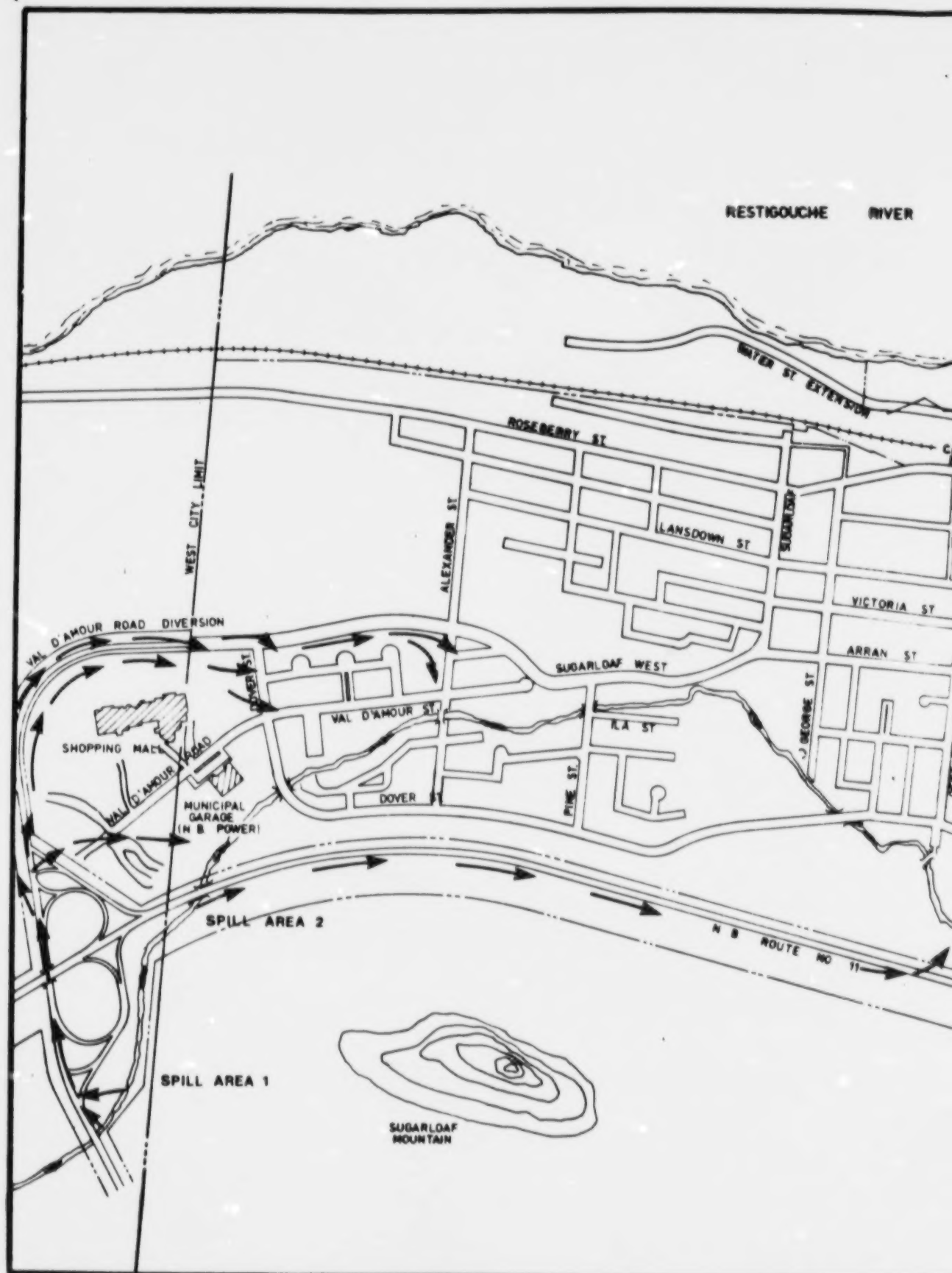


FIGURE 4.2



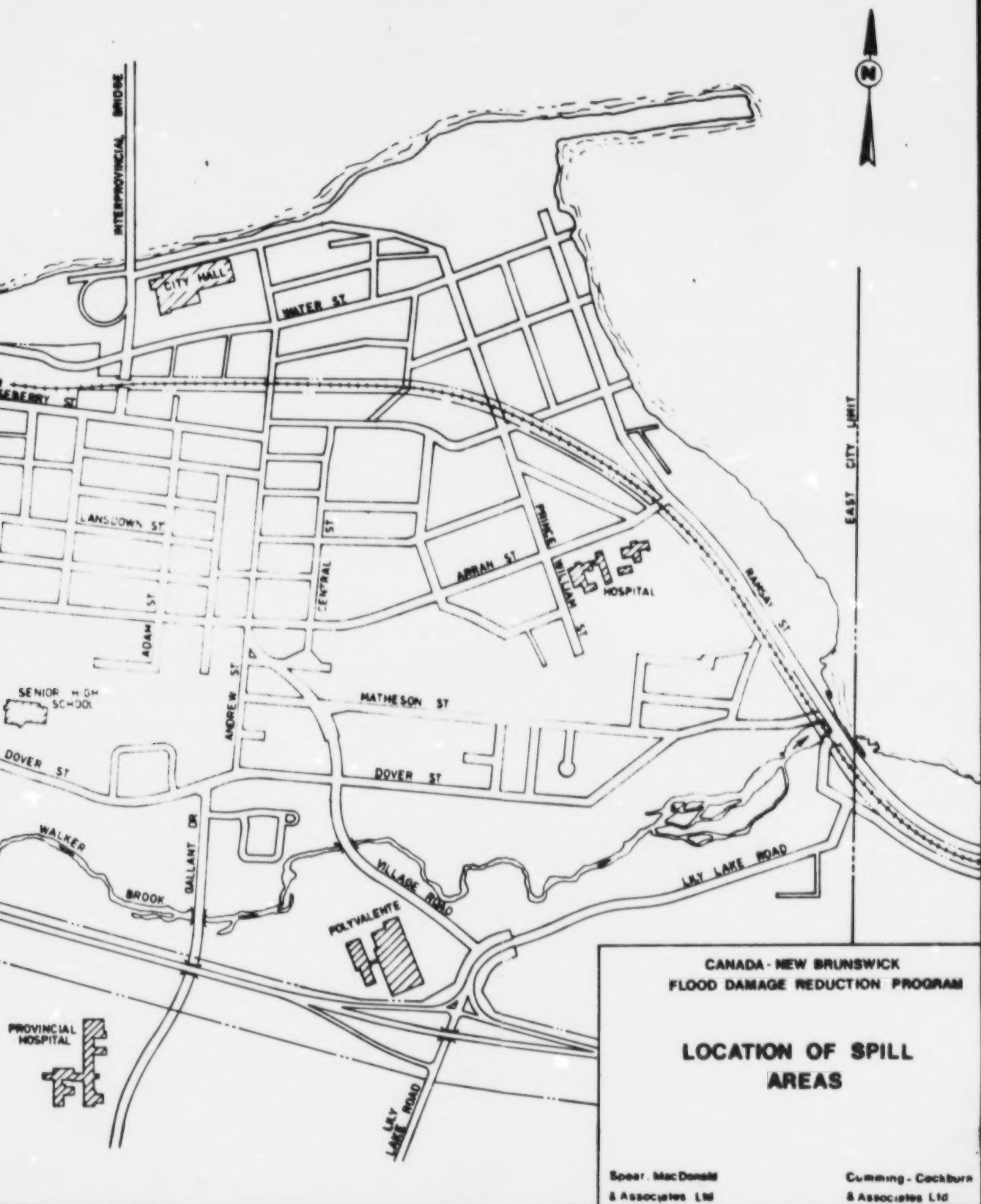
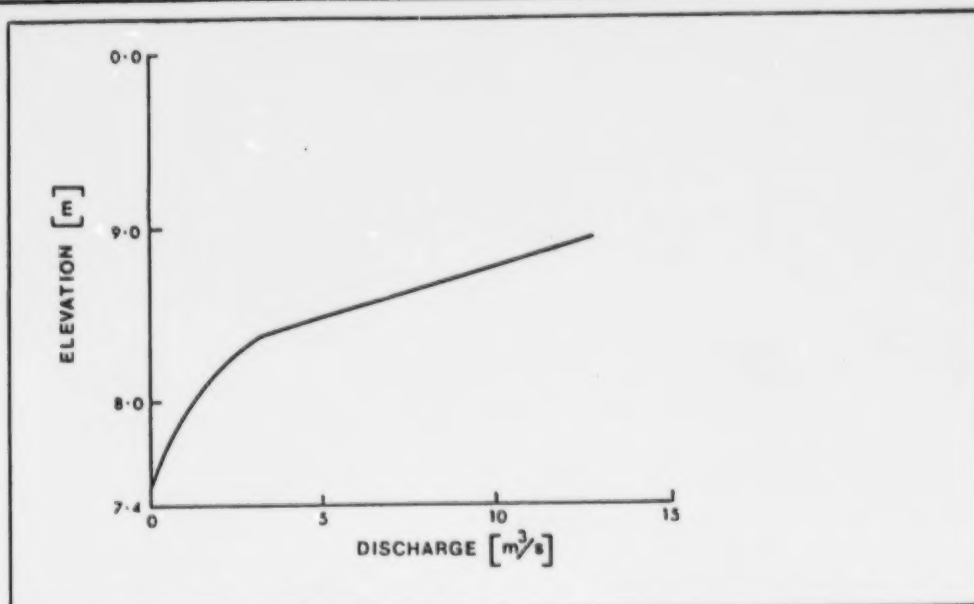
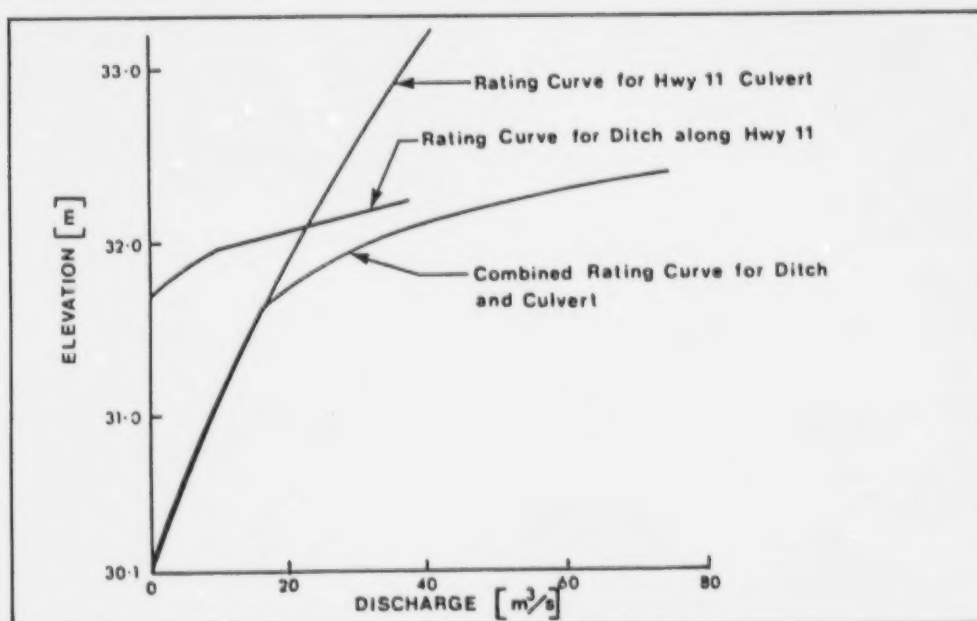


FIGURE 4.3



RATING CURVE FOR SPILL SECTION AT OLD DAM (S26)



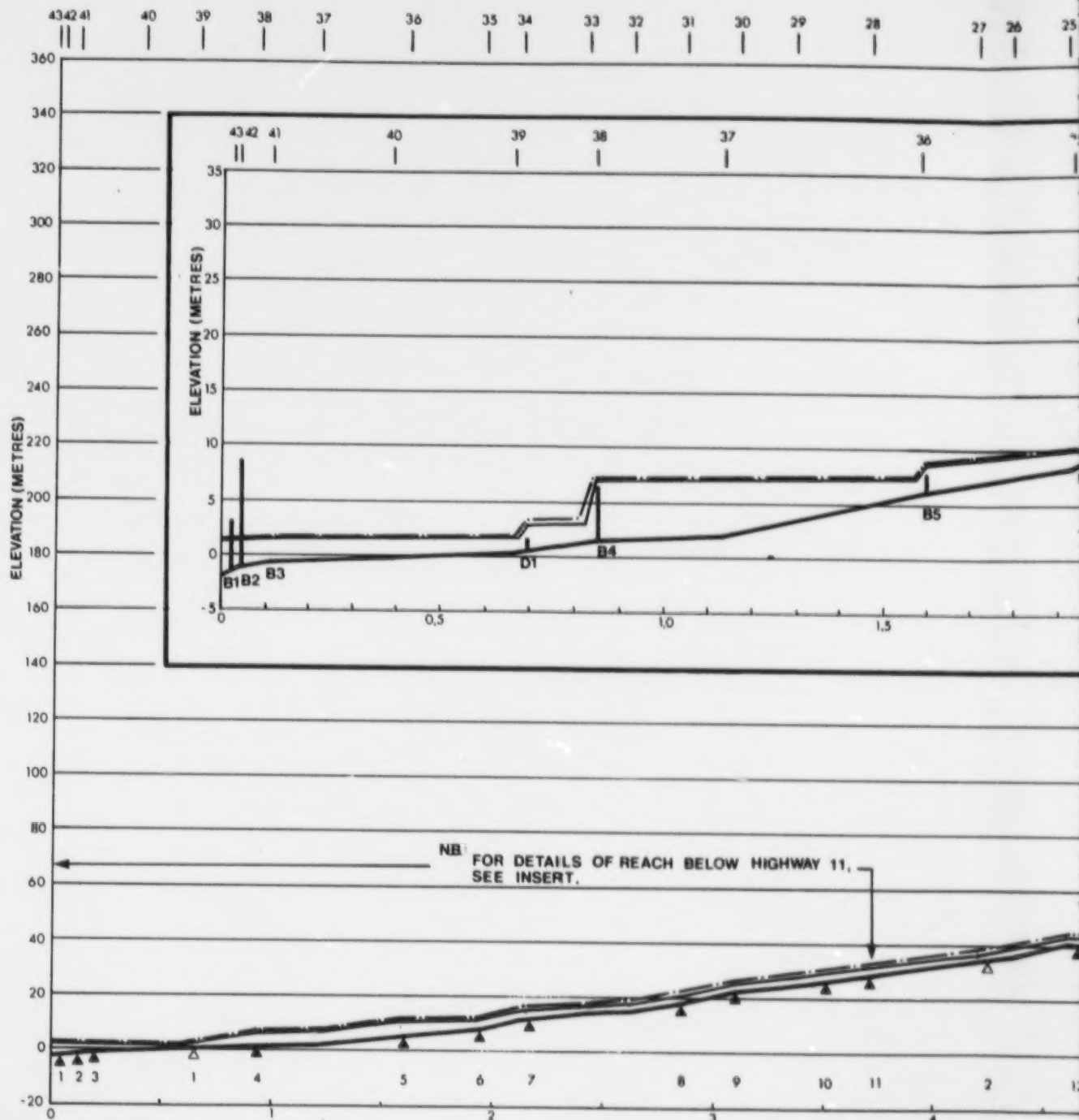
RATING CURVE FOR SPILL AT HWY 11 (B11)

CANADA - NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM

RATING CURVE FOR SPILL SECTIONS

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Cumming - Cockburn
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No. LOCATION

B1	RANSEY ST.
B2	CNR MAIN LINE
B3	WATER MAIN TRUSS
D1	OLD DAM (DOVER & MATHESON)
B4	VILLAGE ROAD
B5	GALLANT DR.
B6	GERRARD ST.
B7	DOVER RD.
B8	IDA & PINE ST.

ELEVATION SOFFIT DECK

2.1	3.0
5.5	8.4
N/A	N/A
1.6	1.6
3.4	6.3
7.0	7.7
N/A	N/A
15.6	16.2
21.6	22.1

No. LOCATION

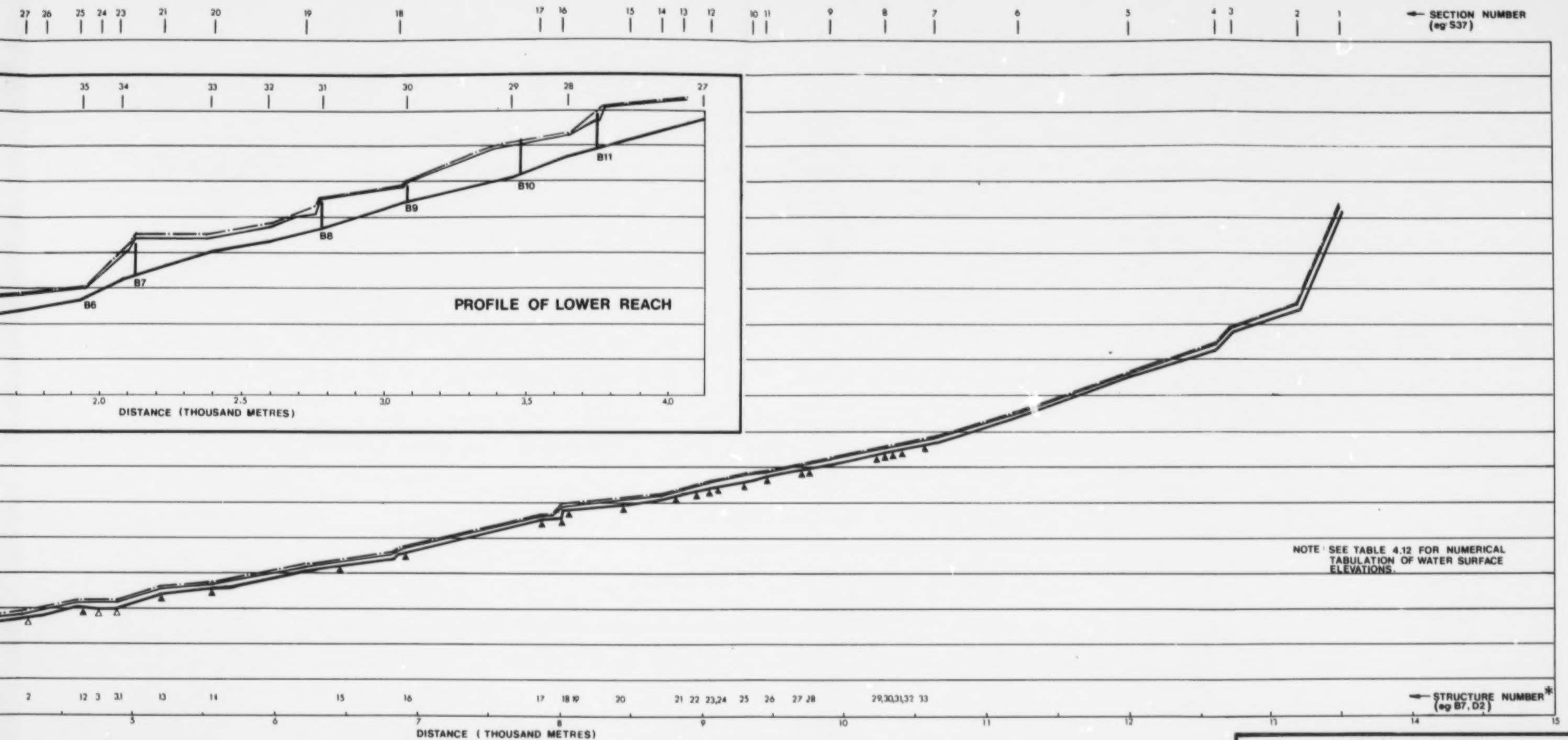
B9	ALEXANDER ST.
B10	DOVER RD. (NBEPC)
B11	HIGHWAY 11
D2	OLD DAM U/S WSC GAUGE
B12	SUGARLOAF PARK
D3	SUGARLOAF PARK WEIRS (1st WEIR)
D3.1	SUGARLOAF PARK WEIRS (2nd WEIR)
B13	SNOWMOBILE CROSSING

ELEVATION SOFFIT DECK

24.2	24.5
29.7	30.9
33.2	34.5
38.2	38.2
41.9	42.9
40.5	40.5
42.3	42.3
51.8	54.8

STRUCTURE

No.	LOCAT
B14	WOOD BR
B15	PRIVATE (MASONRY)
B16	PRIVATE
B17	GULCH RD
B18	GULCH RD
B19	WALKWAY
B20	PRIVATE
B21	ROUTE 27
B22	PRIVATE



CANADA - NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM

HYDRAULIC PROFILES-UNADJUSTED FOR SPILL

LEGEND

— CHANNEL INVERT

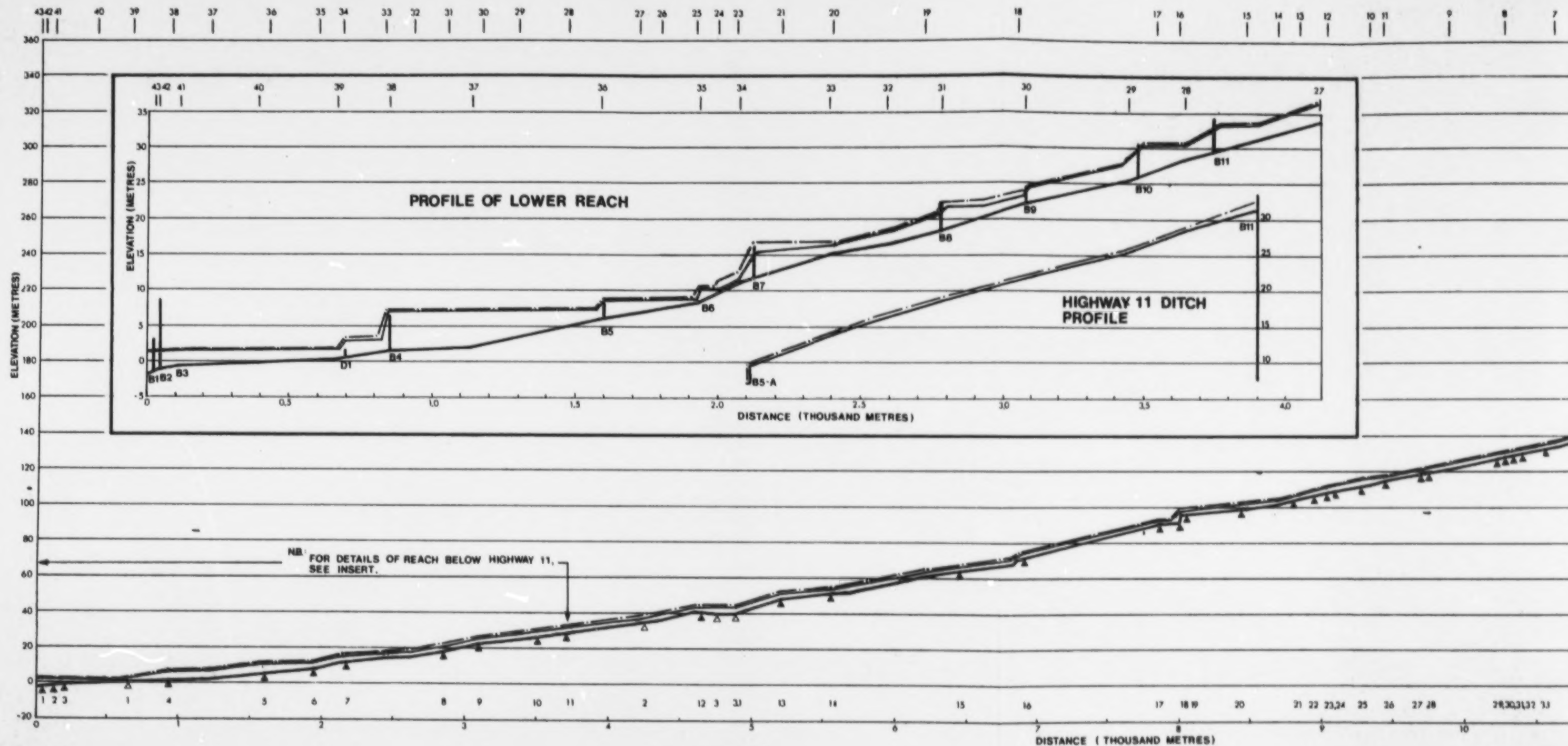
— 1:100 YEAR FLOOD PROFILE

— 1:20 YEAR FLOOD PROFILE

Spear Macdonald
& Associates Ltd

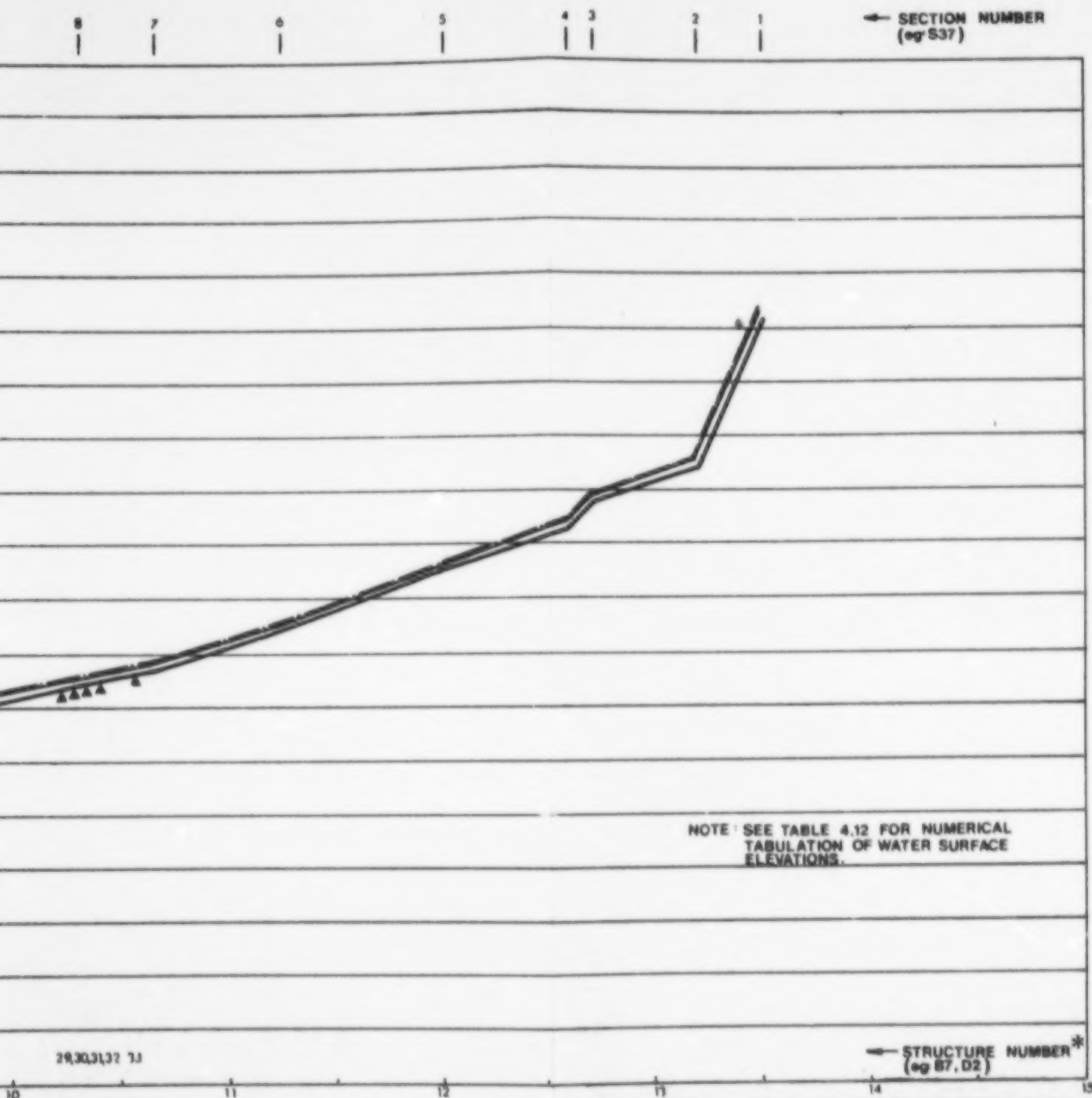
Cumming-Cockburn
& Associates Ltd

FIGURE 4.5



STRUCTURE INDEX*

No.	LOCATION	ELEVATION SOFFIT DECK	No.	LOCATION	ELEVATION SOFFIT DECK	No.	LOCATION	ELEVATION SOFFIT DECK	No.	LOCATION	ELEVATION SOFFIT DECK
B1	RANNEY ST.	2.1 3.0	B9	ALEXANDER ST.	54.2 54.5	B14	WOOD BRIDGE / PRIVATE DR.	54.2 54.8	B25	PRIVATE DR. D/S GRAVEL PIT RD.	108.5 108.8
B2	COR MAIN LINE	5.5 8.4	B10	DOVER RD. (NB/PC)	29.7 30.9	B15	PRIVATE BRIDGE (STA. A NARROW)	69.2 69.5	B24	GRAVEL PIT RD.	110.5 113.4
B3	WATER MAIN TRUSS	N/A N/A	B11	HIGHWAY 11	33.2 34.5	B16	PRIVATE BRIDGE U/S STA. A	74.8 75.1	B28	LILY LAKE RD.	114.0 114.5
B4	OLD DAM (DOVER & NARROW)	1.0 1.6	B12	OLD DAM U/S NSC GAGE	38.2 38.2	B17	GULCH RD.	92.9 93.7	B26	CHURCH BRIDGE	115.5 115.9
B5	GALLANT DR.	3.4 6.3	B13	SUGARLOAF PARK	41.9 42.9	B18	GULCH RD. DAM	93.9 94.2	B27	CMP UNDER RTE. 270 (1st CROSSING)	117.1 117.5
B6	GERRARD ST.	7.0 7.7	B3	SUGARLOAF PARK WEIRS (1st WEIR)	40.5 40.5	B19	WALKWAY U/S GULCH RD.	100.1 100.6	B28	PRIVATE DR. U/S OF CHURCH	117.2 117.6
B7	WATER RD.	15.6 16.2	B3.1	SUGARLOAF PARK WEIRS (2nd WEIR)	42.3 42.3	B20	PRIVATE DR.	100.1 100.4			
B8	2nd & 2nd ST.	21.6 22.1	B13	SNOWBELL CROSSING	51.8 54.6	B21	ROUTE 270 & MALAISE RD.	105.8 106.2			
						B22	PRIVATE DR. U/S MALAISE	105.3 108.7			
									B29	CMP UNDER RTE. 270 (2nd CROSSING)	128.6 129.1
									B30	PRIVATE RD.	132.7 133.9
									B31	PRIVATE RD. TO BODY SHOP	133.0 133.4
									B32	BRIDGE RUINS & BODY SHOP	N/A N/A
									B33	CMP UNDER RTE. 270 (3rd CROSSING)	134.6 136.3



29,30,31,32 7.1

KEY

- 37 | SECTION LOCATION/No. (eg: S37)
 ▲ BRIDGE LOCATION
 Δ WEIR/DAM LOCATION

CANADA - NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM

**HYDRAULIC PROFILES - ADJUSTED FOR
SPILL**

LEGEND

- CHANNEL INVERT
 - - - 1:100 YEAR FLOOD PROFILE
 ——— 1:20 YEAR FLOOD PROFILE

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Cumming-Cockburn
& Associates Ltd

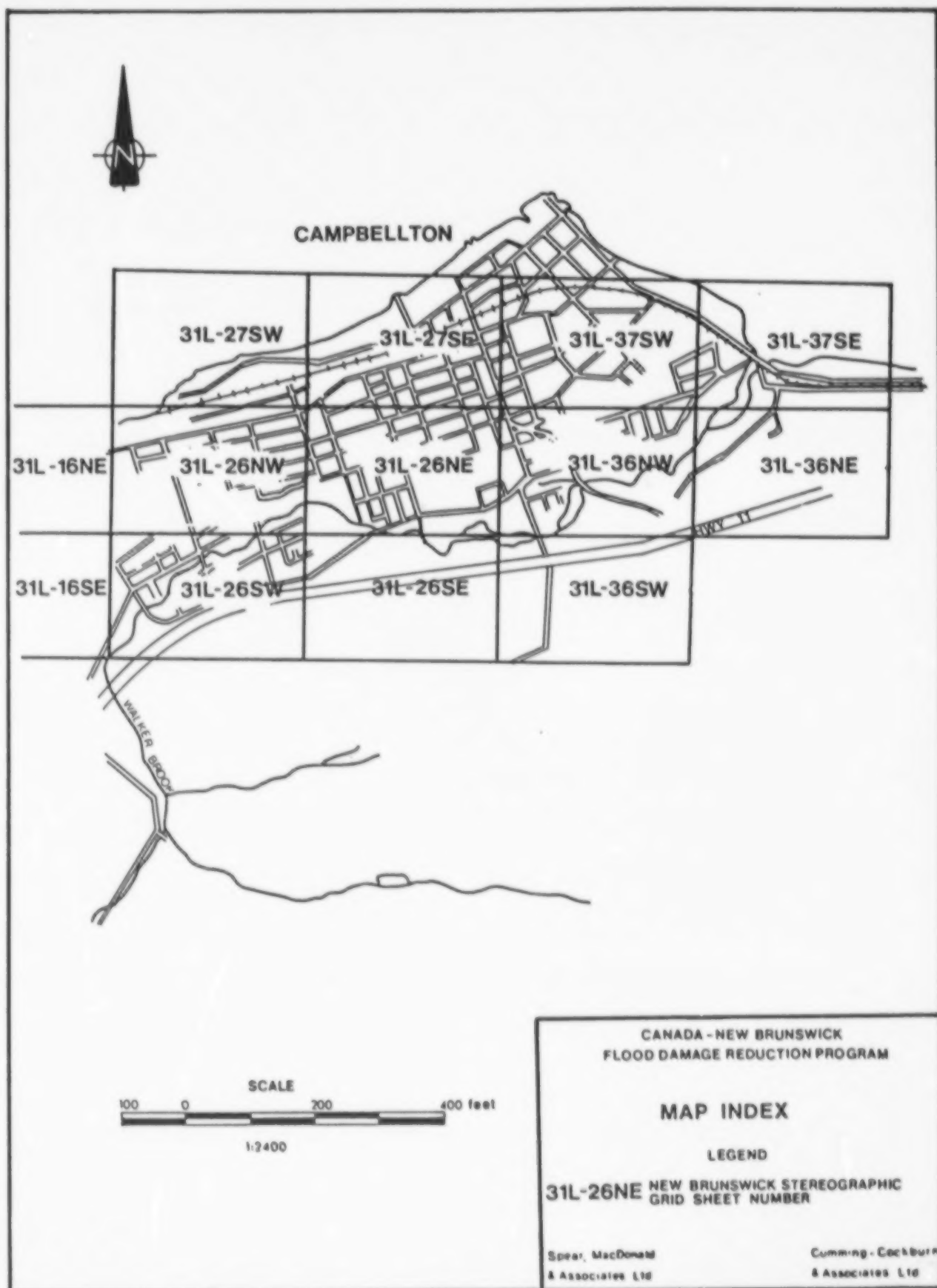


FIGURE 4.7

5.0 REMEDIAL MEASURES

5.1 General

On the basis of the identification of historical flooding and utilizing the results of the backwater model for the 1:20 and 1:100 year floods, it was possible to identify alternative remedial measures for alleviating the flood hazard along Walker Brook.

Broadly speaking, the basic elements for a flood damage reduction plan can be classified as:

1. Structural measures which directly affect the flood characteristics, and
2. Non-structural measures which are intended to modify the loss burden, either by reducing the potential for continued development in flood prone lands or by providing some form of economic relief from flood losses.

A detailed analysis of possible remedial measures was beyond the scope of the present investigations. However, based on the results of the study it has been possible to identify a number of alternative remedial measures for further future detailed consideration.

5.2 Identification of Structural Alternatives

The following structural flood control measures should be considered in order of priority:

1. Flow diversion at Route 11:

The existing natural diversion of flow from Walker Brook along Highway 11 should be investigated further to define the feasibility of designing a permanent flow diversion structure. This should include the following aspects:

- a) An analysis of various means of keeping the ditch free from snow or debris;

- b) The size required for enlargement of the downstream culvert to direct flow from the ditch just upstream of the Gallant Drive area to Walker Brook;
- c) The degree of flow diversion from Walker Brook (i.e. maintain existing "natural" flow diversion rates or increase by lowering the ditch invert);
- d) Additional alterations required to fully contain the reduced flow in the downstream channel;
- e) Other factors.

The diversion of peak flows along Highway 11 effectively reduces the flood levels at all locations downstream to Gallant Drive and would allow the existing channel to contain most of the flow with only minor alterations at some locations.

2. Removal of flow obstructions:

This would include removal of obstructions to flow at the following locations:

- a) Removal of the abandoned dam near Sugarloaf Mountain
- b) The replacement of the culverts at Village Road
- c) The replacement of the Bridge crossing at Gallant Drive
- d) The removal of the abandoned dam downstream of Village Road.

3. Dykes or flood berms along selected reaches of the channel:

The feasibility of constructing a low flood control berm along the stream reach immediately downstream of Highway 11 should be considered. About 500 m of berm might be required along the northwest bank of the stream down to just below Dover Street. Some berming would also be required in the southeast bank just upstream of Dover Street. The possibility of raising Dover Street to minimize spill to the downstream residential areas should also be investigated together with the possibility of enlarging or twinning the Dover Street Culvert.

4. Additional stream channel improvements such as:
- a) Clearing of vegetation from the channel in the area downstream of Village Road
 - b) Dredging of gravel and cobbles from the channel from above Gallant Drive to downstream of Village Road
 - c) Straightening portions of the channel downstream of Dover Street to Gallant Drive, particularly in the vicinity of Gerrard Street.
 - d) Channel enlargement (e.g. in the area from just downstream of Highway 11 to just downstream of Dover Street).

After the provision for overflow along Highway 11 has been analysed in detail, the effects on minor culvert improvements at several locations on backwater in the upstream areas should also be considered. This includes minor culvert improvements at the following locations:

- 1) Pine and 11a Street (B8)
- 2) Alexander Street (B9)
- 3) Dover Road (NBEPC) (B10)

These investigations would include means to improve the efficiency of existing culverts at these locations and the effect on reducing flood levels just upstream.

The most attractive alternatives presently appear to be diversion of some flow along Highway 11 together with replacement of the culverts at Village Road and the bridge at Gallant Drive. The latter remedial measures would also reduce the potential for ice and debris jams at these locations. The benefits of flood damage reduction should be assessed by estimating the potential flood damages associated with the occurrence of various flood events.

5.3 Non-structural Flood Control Measures

In developing areas such as along Walker Brook, floodplain regulations could be implemented to restrict future development and thus reduce the potential for continued increases in flood damages. In this case a two-zone floodway flood-fringe concept is envisaged where zoning regulations would prohibit future development in the high hazard areas. Additional development might be permitted

in the flood-fringe areas, depending on the degree of hazard and the implementation of flood proofing measures to protect these developments. Other non-structural measures which might be considered include relocation of flood prone structures. This could involve expropriating high hazard developments and prohibiting future development in these areas.

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APPENDIX A

BACKGROUND FOR

STATISTICAL HYDROLOGIC ANALYSES

FREQUENCY ANALYSES

APPENDIX A

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APPENDIX A : BACKGROUND FOR STATISTICAL ANALYSES

A.1 REGIONAL FLOOD FREQUENCY ANALYSIS

The following sections provide additional information describing the methodology applied to the Statistical Analyses of the Walker Brook watershed.

A.1.1 Statistical Analysis on Eel River

A.1.1.1 Testing the Data Base

As outlined in Section 2.2.1 of the report, the sample data set of maximum instantaneous flows was tested for stationarity and randomness, using graphical techniques. A summary of the sample extreme-value data set available for the Eel River at Eel River Crossing is given in Table A.1. The sample data set was tested for stationarity and randomness using the following technique:

- a) A progressive mean of annual flood peak (m^3/s), (Figure A.1), i.e.

$$QP_n = \frac{1}{n} \sum_{i=1}^n Q_{Pi} \quad (A.1)$$

where : QP_n = cumulative moving mean max. instantaneous peak flow at
year n

i = year, 1 to n

Q_p = annual max. instantaneous flood peak (m^3/s)

- b) Temporal distribution of QP (Figure A.2)

where : Q_p = annual max. instantaneous flood peak (m^3/s)

The study of these data plots reveal that the data can be regarded as stationary (i.e. no trend or persistence is evident from the graphical analyses). There is also no evidence of non-stationarity in the data set. The data plots are presented in Figures A.1 and A.2.

A.1.1.2 Flood Frequency Analysis

Several distributions were fitted to the available Eel River data set for comparison purposes, including Gumbel, Log Normal (LN), Three Parameter Log Normal (3PLN) and Log Pearson Type III (LP3). Table A.2 presents a summary table of the different distributions and the corresponding standard errors.

For the North region of New Brunswick, where the data set is less than 20 years and the coefficient skew is equal to 0, Acres (Acres, 1977) recommends that the log normal distribution be used. However, in this case, the log normal distribution yields the lowest flow estimates. Using a visual comparison, both 3 parameter distributions gave a good fit (see Figure A.3) for the given data set. The peak flow estimates for the LP3 distribution are also slightly higher than the corresponding flows using the LN distribution, but lower than flow estimates using the 3PLN distribution. Comparing the theoretical statistics of skew and kurtosis for the various distributions with those estimated from the data set, the LN and 3 PLN were found to yield close agreement. A comparison could not be made with the LP3 as this distribution generates the estimates for each data set. Between the two 3 parameter distributions, the LP3 has the lowest standard error. This results in the flow estimates at the +95% confidence limits being less than those for the 3PLN distribution. (On the other hand, for the -95% confidence limits the 3PLN distribution gives a lower peak than the LP3 distribution.) On the basis of best fit, correlation to theoretical statistics and due to the fact that the resulting flow estimates are intermediate when compared with other distributions, the LP3 distribution was selected for use in this study.

The flood frequency plot using the "best fit" distribution is presented in Figure A.3.

A.1.2 Regional Flood Frequency Analysis on Walker Brook

The Regional technique used for calculating the 1:20 and 1:100 year instantaneous flood peaks assumes the following equations:

95% confidence limits

$$Q_{P20} = 1.12 A^{0.861} W^{5.47} F_{DTS}^{-0.920} \quad (+48\%, -32\%) \quad (A.2)$$

$$Q_{P100} = 1.48 A^{0.865} W^{6.42} F_{DTS}^{-1.08} \quad (+60\%, -37\%) \quad (A.3)$$

where A = Area (km^2)

$$W = (A - A_s) / A$$

F_{DTS} = Proportion of annual maximum flood peaks occurring
in the March, April, May period at the drainage basin centroid

A_s = area swamps and lakes (km^2)

The applicable range of Independent Variables and the values assumed for Walker Brook are presented in Table A.3.

The design flow estimates for the Eel River and the Walker Brook are presented in Table A.4.

Using the ratio of the calculated flood peaks at the two sites, the gauged values were converted to the desired design flood peaks by applying the following equations:

$$Q_T = Q'_T (Q_{CT}/Q'_{CT})$$

where Q_T = desired design flood peak of return period T-years
in m^3/s

Q'_T = flood peak of return period T obtained from nearby flood
frequency curve on Eel River (m^3/s)

Q'_{CT} & Q_{CT} = flood peaks calculated using regional technique at the
location of interest and the gauge site respectively
(m^3/s)

The resulting estimated maximum instantaneous flow rates for Walker Brook at
the Water Survey of Canada gauge location are presented in Table A.5.

TABLE A.1
Summary of Eel River Data Set

EEL RIVER NEAR EEL RIVER CROSSING
01BJ004

Year	Maximum Instantaneous Discharge (m ³ /s)	Maximum Daily Discharge (m ³ /s)
1967	N/A	N/A
1968	34.5	30.6
1969	40.5	35.7
1970	60.9	47.3
1971	34.5	26.7
1972	25.0	23.1
1973	44.7	40.8
1974	32.6	28.2
1975	36.2	32.8
1976	21.3	20.3
1977	23.3	20.6
1978	26.6	22.4
1979	47.6	41.1
1980	35.1	30.6
1981	20.9	20.3

NOTES: 1) N/A - not available
2) Source: Water Survey of Canada

TABLE A.2
Estimates and Standard Error
for Various Distributions Tested

Discharge Data

EEL RIVER - STATION NO. 01B J004

YEAR	DATA	ORIG. REC'D	NANA	PROP.	RET. PERIOD
1968	34.500	40.900	1	.067	15.000
1969	40.500	47.600	2	.133	7.500
1970	40.900	44.700	3	.200	5.000
1971	34.500	40.500	4	.267	3.750
1972	25.000	36.700	5	.333	3.000
1973	44.700	35.100	6	.400	2.500
1974	32.600	34.500	7	.467	2.143
1975	36.200	34.500	8	.533	1.875
1976	21.300	32.600	9	.600	1.667
1977	21.300	26.600	10	.667	1.500
1978	26.600	25.000	11	.733	1.364
1979	47.600	23.300	12	.800	1.250
1980	35.100	21.300	13	.867	1.154
1981	20.900	20.900	14	.933	1.071

Statistics

EEL RIVER - STATION NO. 01B J004

SAMPLE STATISTICS

MEAN = 35. S.D. = 11.3 C.S. = .0030 C.K. = 4.6704

SAMPLE STATISTICS (LOGS)

MEAN = 3.4953 S.D. = .3168 C.S. = .1948 C.K. = 3.3108

SAMPLE MIN = 21. SAMPLE MAX = 61. N = 14

PARAMETERS FOR GUMBEL I A = .117904 U = 30.

PARAMETERS FOR LOGNORMAL N = 3.4953 S = .3168

STATISTICS OF LOG(X-A)

MEAN = 2.9052 S.D. = .5636 C.S. = -.2128 C.K. = 3.0540

Flood Frequency Distributions

PARAMETERS FOR THREE PARAMETER LOGNORMAL A = 14. N = 2.9052 S = .5636
PARAMETERS FOR LOG PEARSON III BY MOMENTS A = .0309 B = .1054E+03 LOG(N) = .2427 H = .1275E+01
NO MAXIMUM LIKELIHOOD SOLUTION FOR LOG PEARSON III

GUMBEL I			LOGNORMAL		THREE PARAMETER LOGNORMAL		LOG PEARSON III MAX. LIKELIHOOD MOMENTS			
RETURN PERIOD	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT	FLOOD ESTIMATE	ST. ERROR PERCENT
1.050	15.400	9.50	14.600	12.60	17.800	12.70	0.000	0.00	15.400	23.40
1.050	20.100	12.80	19.600	13.00	20.800	8.54	0.000	0.00	19.900	12.80
1.250	25.500	8.89	25.200	9.84	24.900	9.18	0.000	0.00	25.200	9.39
2.000	32.700	8.15	33.000	8.47	31.800	9.17	0.000	0.00	32.600	9.17
5.000	42.300	7.44	43.000	9.84	42.900	12.00	0.000	0.00	42.900	10.40
10.000	48.600	10.80	49.500	11.40	51.200	15.40	0.000	0.00	49.800	12.70
20.000	54.700	11.70	55.500	13.00	59.700	19.30	0.000	0.00	56.500	15.90
50.000	62.600	12.70	63.200	14.90	71.700	24.90	0.000	0.00	65.100	21.10
100.000	68.600	13.40	68.900	16.30	91.300	29.10	0.000	0.00	75.100	25.50
200.000	74.500	13.90	74.500	17.60	91.600	33.30	0.000	0.00	79.000	30.10
500.000	82.200	14.50	82.100	19.20	106.000	38.80	0.000	0.00	88.500	36.50

TABLE A.3
Independent Variables

	<u>Area (km²)</u>	<u>(proportion)</u>	<u>FDS (proportion)</u>	<u>Slope (percent)</u>
Applicable Range	25-5000	.85 - 1.0	.45 - 1.00	.08 - 1.57
Walker Brook at outlet	67.4	.99	1.00	1.79
At Sugarloaf Mountain	61.9	.98	1.00	1.86
Upstream of Sugarloaf Mountain	45.0	.99	1.00	1.98

TABLE A.4
Design Flow Estimates Using Regional Techniques

<u>Watercourse</u>	<u>1:20 Year Peak (cms)</u>	<u>1:100 Year Peak (cms)</u>
Eel River (at gauge)	52.6	70.5
Walker Brook at Outlet	39.1	51.8
Walker Brook at Sugarloaf Mountain	36.1	47.8
Walker Brook upstream of Sugarloaf Mountain	28.6	38.2

TABLE A.5
Maximum Instantaneous Flow Rates - Walker Brook
(Incorporating Eel River Flood Frequency Analysis)

<u>Location</u>	Drainage Area (km ²)	Max. Instan. Discharge (cms)	
		<u>1:20</u>	<u>1:100</u>
At Outlet	67.4	42.0	53.0
At Sugarloaf Mountain	61.9	38.8	48.9
Upstream of Sugarloaf Mountain	45.0	30.7	39.1

CANADA-NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM

EEL RIVER

CUMULATIVE MOVING MEAN FOR TREND

Spear, MacDonald
& Associates Ltd

Cumming-Cuthbertson
& Associates Ltd

Eel River near Eel River Crossing
01BJ004
1967-1981, N=14

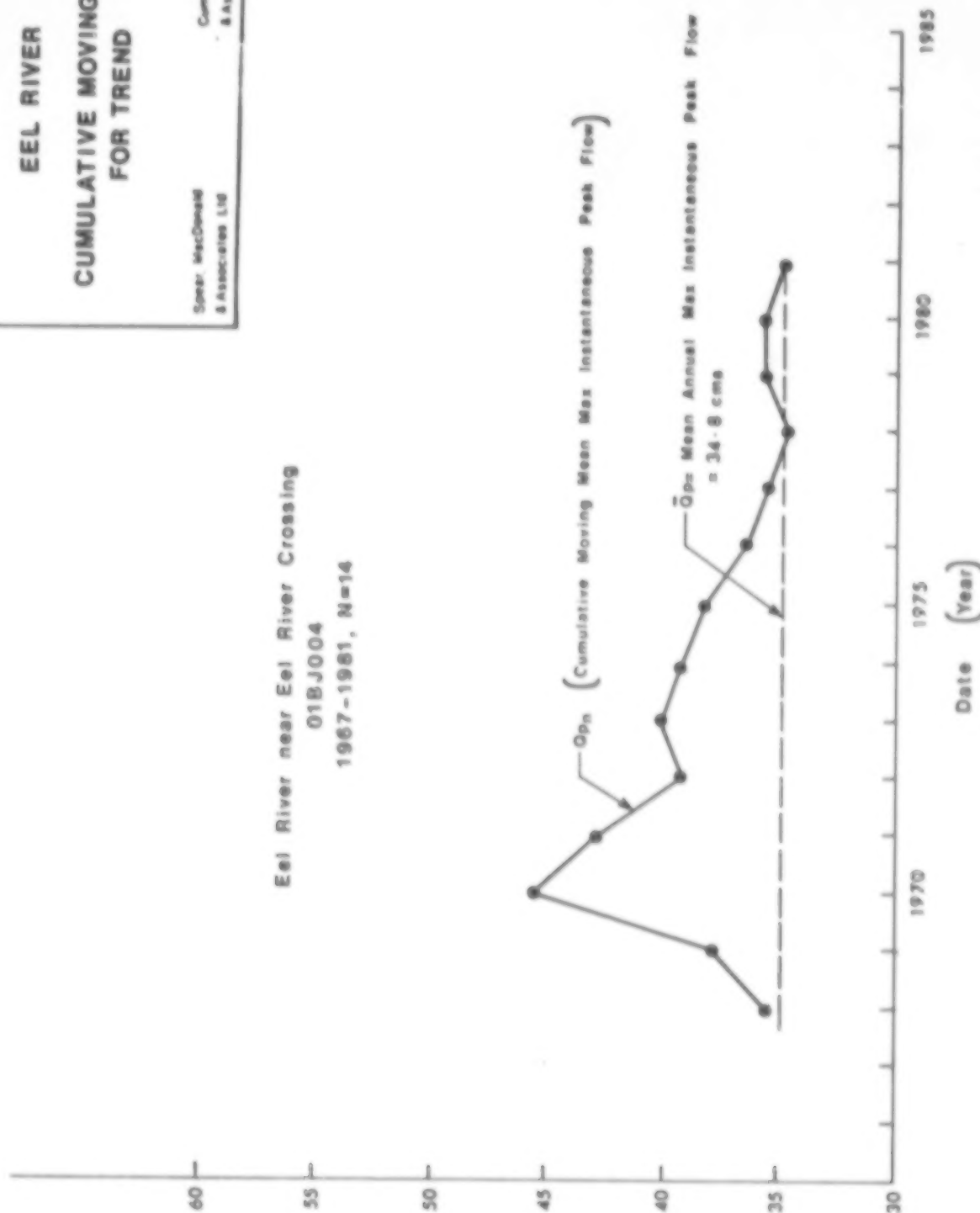
Mean Discharge Q_n (cms)

Q_n (Cumulative Moving Mean Max Instantaneous Peak Flow)

$\bar{Q}_n$ Mean Annual Max Instantaneous Peak Flow
 ≈ 34.8 cms

Date (Year)

FIGURE A.1



CANADA-NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM

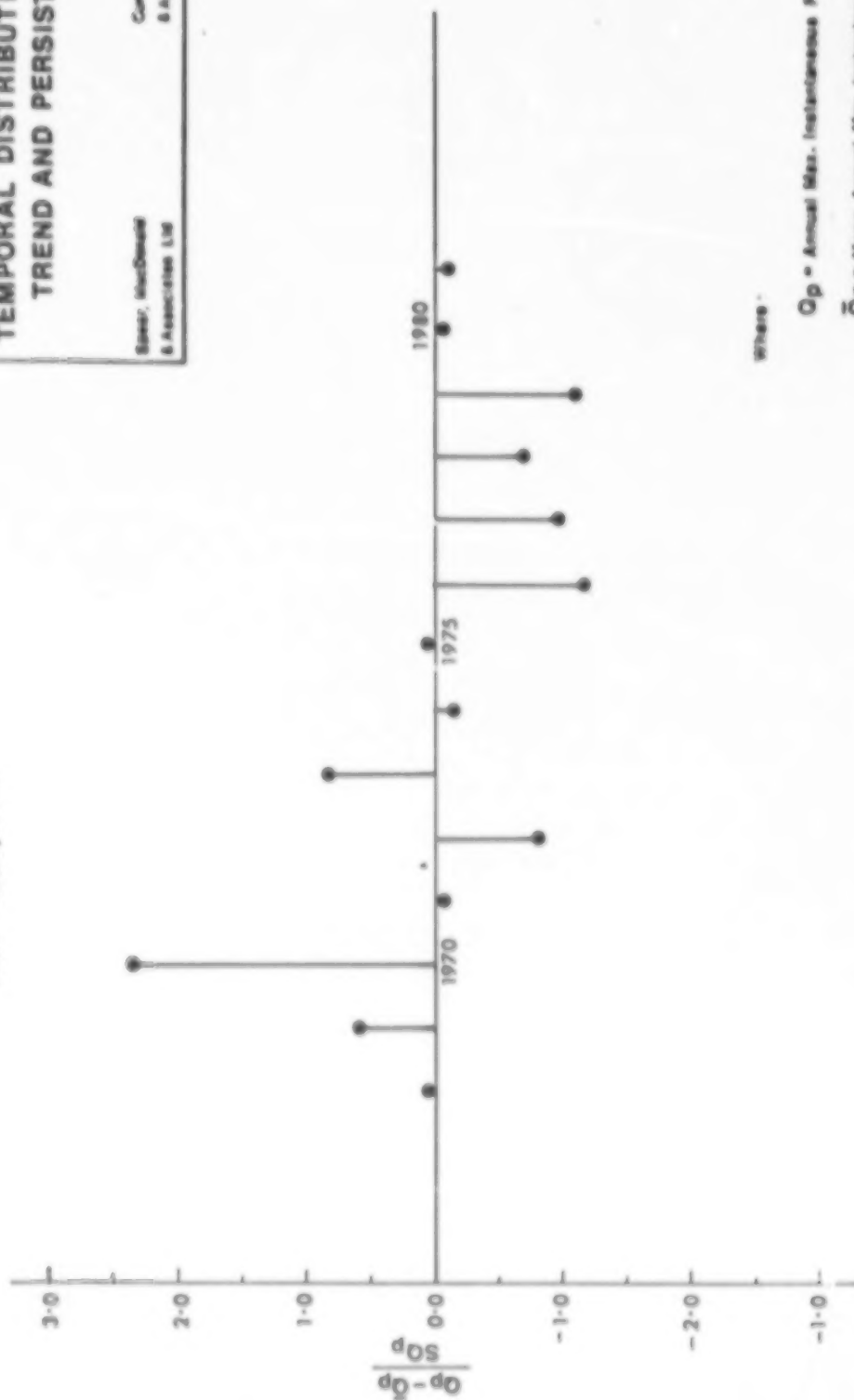
EEL RIVER

TEMPORAL DISTRIBUTION FOR
TREND AND PERSISTENCESpeer, MacDonald
& Associates LtdCumming-Cochburn
& Associates Ltd

Eel River near Eel River Crossing

01BJ004

1967-1981, N-14



where

 Q_p = Annual Max. Instantaneous Peak Flow $\bar{Q}_p$ = Mean Annual Max. Instantaneous
Peak Flow = 34.8 cmg SQ_p = Standard Deviation

FIGURE A.2

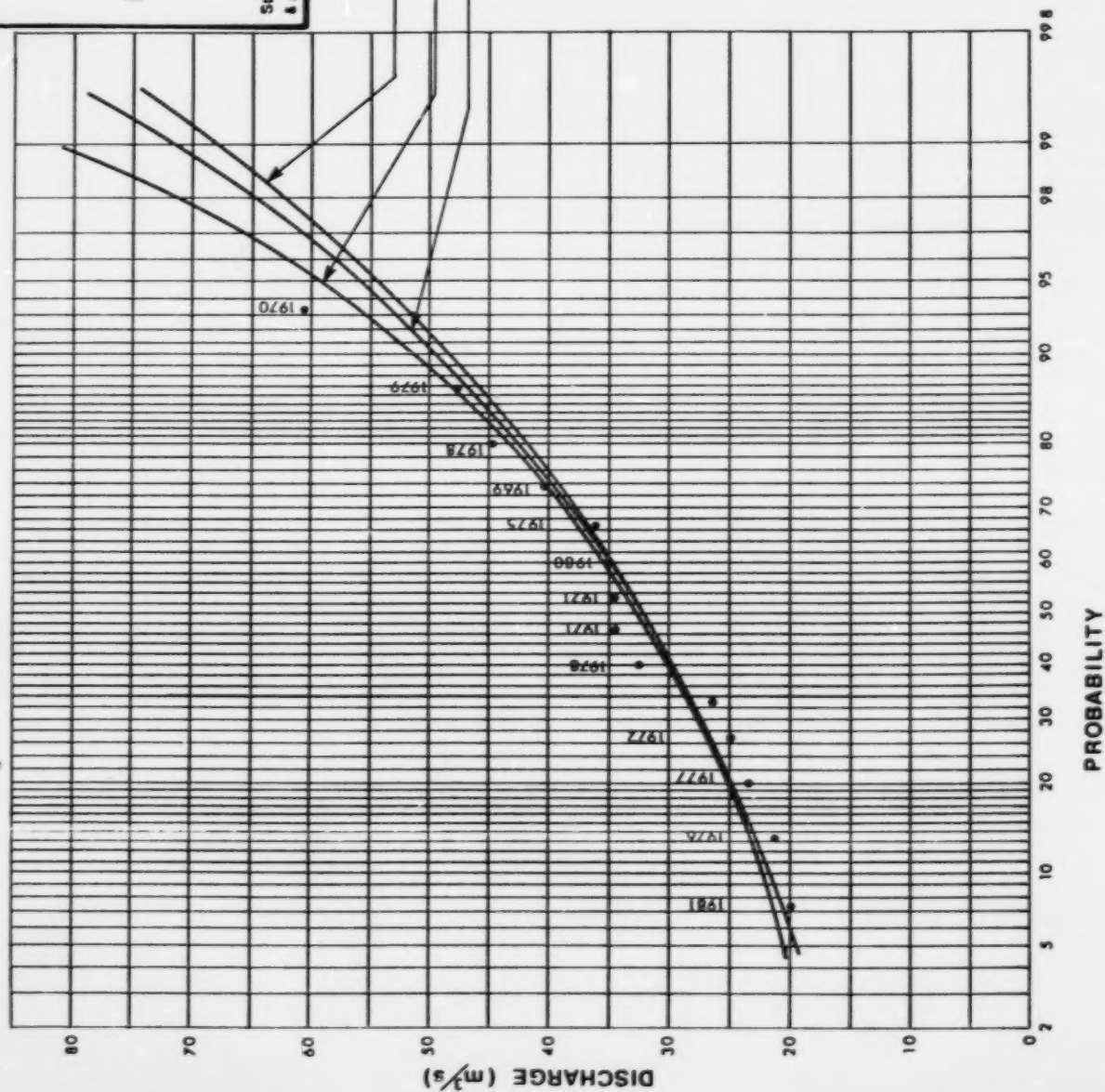
CANADA - NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM

EEL RIVER

FLOOD FREQUENCY ANALYSIS

Spear, MacDonald
& Associates Ltd

Cumming-Cockburn
& Associates Ltd



SAMPLE STATISTICS			
	X	LOG X	LOG(X-A)
MEAN	35	3.4953	2.9052
S.D.	11.3	0.3168	0.5636
C.S.	0.8830	0.1948	-0.2158
C.K.	4.6704	3.3108	3.0548

APPENDIX B

BACKGROUND FOR

DETERMINISTIC ANALYSES

APPENDIX B

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APPENDIX B : BACKGROUND FOR DETERMINISTIC ANALYSES

The following sections provide a more comprehensive overview of the methodology applied in determining the parameters for use in the HYMO model regarding the Deterministic Analyses of the Walker brook watershed.

B.1 Hydrological Input Parameters

In order to assess the response of the watershed to a rainfall event, the time of concentration and recession constant for the unit hydrograph must be determined. The following relates the various methods undertaken in order to develop these parameters.

B.1.1) Time to Peak and Recession Constant

- a) The assessment of the time to peak parameter, T_p , for each subwatershed involved the application of the following equation:

$$T_p = 4.63 A^{.422} SLP^{-.46} (L/w)^{.133} \quad (B.1)$$

where : A = area of watershed (mi^2)
 SLP = slope of watershed (ft/mi)
 L = length of watershed (mi)
 w = width of watershed (mi)

- b) The assessment of the other parameter, that being the basin recession constant, K , involved the application of the following equation as given in the publication on the program HYMO:

$$K = 27.0 A^{.231} SLP^{-.777} (L/w)^{.124} \quad (B.2)$$

where A = watershed area (mi^2)
 L = watercourse length (mi)
 w = watershed width (mi)
 SLP = slope of watercourse (ft/mi)

Results of these analyses are summarized in Table B.1.

B.1.2) Model Calibration

Utilizing the available meteorological data from recording stations in the area and streamflow records from the Walker Brook station, the HYMO model was calibrated to summer storm flood events and spring melt flood events.

Due to the absence of any flood events of significant magnitude, the results of the model calibration are not completely satisfactory. While the time to peak (T_p) of the watershed was adequately represented, the peak flow is simulated with some error.

For the calibration of the parameters that determine the shape of the hydrograph, namely K and T_p values, a trial and error approach was adopted. The resulting subwatershed hydrograph parameters are presented in Table B.2. These parameters were used in estimating flows for the various design events.

B.1.3) Soil Cover Complex Number

The soil cover complex number for each of the subcatchments which comprise the Walker Brook watershed, was calculated based on the method outlined by the Soil Conservation Service.

As discussed previously in this report, weighted soil curve numbers were established for each of the 12 sub-basins. Weighted curve numbers as given in Table 3.7, located in section 3.3.4 of this report, were calculated using the values shown on Table B.3.

B.1.4 Reservoir Characteristics

As described in the report, the man-made reservoirs within the watershed were modelled using the storage indication method available as a subroutine in the HYMO computer model. The reservoirs modelled include Pritchard Lake, Smith Lake and the large unnamed lake downstream of Smith Lake. Field measurements were obtained for the outlet structures and topographic maps were used to obtain the available storage. The reservoirs were assumed to be full at the start of the design event simulations. Table B.5 presents the outflow-storage characteristics of each reservoir used in the HYMO model.

B.2 Rainfall Analysis

Charlo A. is the closest principal (i.e. rate of rainfall) climatological station to the Walker Brook watershed (approximately 25 km east) and is situated approximately in the same relationship to the major topographic features of the area. Thus the rainfall data, including the rate of rainfall, should be reasonably representative of conditions in the study basin.

A number of ordinary (i.e. daily data) climatological stations are in operation in the vicinity of the Walker Brook basin. A summary of 24-hour rainfall amounts for these ordinary stations are presented in Figure B.1. The 95% confidence limits for the various stations are also shown. It can be seen that the design amounts for the principal station fall within the 95% confidence limits of all the ordinary climatological stations in the vicinity.

In recent years, two significant rainfall events have occurred in close proximity to, although not in the Walker Brook watershed. Hurricane Belle occurred during August, 1976 and was centred near Edmunston. A second major event was centred near Plaster Rock and although not a hurricane, resulted in higher rainfall amounts than Hurricane Belle.

Because a detailed examination of the August, 1981 event has not been carried out, the storm could not be modelled accurately with the HYMO model. However, a detailed analysis of Hurricane Belle has been undertaken and this event was modelled assuming the event was centred over the Walker Brook watershed. The peak flow generated for this event was greater than the 1:100 year event assuming the Charlo A. 1:100 year rainfall amounts. It would be expected that the August, 1981 event would produce higher flood magnitudes if the storm were centred over the Walker Brook watershed.

The impact of significantly higher rainstorm amounts on peak flow estimates was also analysed. The upper limit for the Probable Maximum Precipitation event (PMP) for the study area in a 24-hour period was determined to be about 390 mm (Atmospheric Environment Services (1981)). This is about four times the 1:100 year rainstorm determined for the study area and is assumed to be

the upper limit for precipitation in the study area. The PMP event was modelled assuming the event was centred over the Walker Brook watershed.

A summary of the 1:100 year Recorded Events and Probable Maximum precipitation amounts for all rainstorms considered is presented in Table B.6.

When modelling the design rainfall events (1:20 and 1:100 year) a storm mass curve derived by A.E.S. was utilized along with the Charlo A. climatological records.

The A.E.S. distribution was selected as it was based upon actual recorded events in the Maritime Provinces. The A.E.S. distribution was derived for both 1 and 12 hour design events. However, because the two mass curves, when converted to dimensionless curves, are very similar, it has been suggested that they may be applicable to other duration events (Dillon, 1980).

The climatological records at Charlo A. were selected for a number of reasons. This station is the only principal climatological station in close proximity to Walker Brook. The data was recommended by A.E.S. (AES, 1981) in a report prepared for the Walker Brook basin. Furthermore, the values used fall within the 95% confidence limits of all other climatological stations in the area.

It can be concluded that the design storms are highly variable with respect to the climatologic station used. For this reason, the design storm approach does not appear to be the most attractive method of design flow estimation.

B.3 Snowmelt Analysis

The secondary snowmelt analysis comprised of using a detailed energy budget equation represented by the following equation:

$$M = M_{RS} + M_{RL} + M_{CC} + M_P + M_G$$

where M = snowmelt

M_{RS} = melt due to short wave radiation

M_{RL} = melt due to long wave radiation

M_{CC} = melt due to condensation and convection

M_P = melt due to rainfall

M_G = melt due to heat transfer from the ground

Utilizing the rainfall intensities at Charlo Airport for the available years of record (1967 - 1980), a frequency analysis of the rainfall amounts for various durations was undertaken and Rainfall-Intensity-Duration curves were established for estimating design rainfall events which could occur in the months of April/May. The curves on Figure B.2 represent the results of the rainfall analysis.

Where possible, other theoretical meteorologic variables were estimated (Gray 1970). The above mentioned equation could then be reduced to a function of 3 meteorologic variables (rainfall, mean daily temperature and wind velocity). Because these variables are not independent, the probabilities of each occurrence associated with a design melt plus rain event is not easily calculated. Hence, this analysis could only be used as a secondary check of the 24 hour rain plus melt amounts calculated using actual events.

Substituting the total 24 hour spring rainfall amounts into the detailed energy budget equation further reduced the equation to a function of two variables.

The Canadian Climatic Normals were also used to obtain typical wind speeds and daily mean temperatures for the April/May period. The values of temperature and wind speed are presented in Table B.7. It was found that the average values, when substituted into the reduced energy budget equation with the 1:100 year rain, produce the 1:100 year rain plus melt. The selection of average values was somewhat arbitrary. However, as a secondary check, it demonstrates that realistic meteorologic data can be combined to produce the design melt plus rain amounts.

TABLE B.1

Summary of Computed T_p and K Values*

<u>Basin No.</u>	<u>HYMO 3 Parameters</u>	
	<u>T_p</u>	<u>K</u>
100	.91	.90
101	.81	.88
110	.92	.84
120	.85	.89
121	.97	.79
102	1.05	1.14
130	.70	.65
140	.65	1.19
141	1.04	1.57
142	.53	.56
143	.45	.53
103	.84	.86

* Parameter values are initial estimates from the empirical equations B.1 and B.2

TABLE B.2
Summary of Calibrated T_p and K Values (1)

<u>Basin No.</u>	<u>Calibrated Parameters</u>	
	<u>T_p</u>	<u>K</u>
100	2.72	2.65
101	2.44	2.64
110	2.47	2.51
120	2.54	2.68
121	2.90	2.37
102	3.15	3.42
130	2.11	1.96
140	1.95	3.57
141	3.12	4.71
142	1.59	1.52
143	1.34	1.59
103	2.51	2.59

- (1) Trial and error model calibration was undertaken in order to adjust the initial parameter estimates given in Table B.1

TABLE B.3
Summary of Typical Runoff Curve Numbers

<u>Landuse</u>	Hydrologic Soil Group*						
	A	AB	B	BC	C	CD	D
Residential	77	81	85	88	90	91	92
Row Crop	62	67	71	74	78	79	91
Woods	45	55	66	71	77	80	83
Pasture	49	59	69	74	79	82	80

* Based on Antecedent Moisture Condition II

TABLE B.4
Computed Subwatershed Curve Numbers

<u>Sub-Watershed</u>	<u>CN (AMC II)</u>
100	78
101	77
110	70
120	78
121	75
102	77
130	71
140	93
141	66
142	78
143	74
103	86

TABLE B.5

Reservoir Characteristics

<u>Reservoir</u>	<u>Outflow (m³/s)</u>	<u>Storage (m³)</u>
Pritchard Lake	0	0
	.16	4196
	.46	8638
	.84	13451
	1.30	18510
	2.89	23816
	5.43	29616
	8.61	35540
	12.30	41833
Smith Lake	0	0
	5.18	9008
	1.47	18263
	2.69	27765
	4.15	37514
	6.61	47386
	9.91	57504
	13.80	67747
	18.21	78359
Unnamed Lake	0	0
	.54	15302
	1.53	30850
	2.80	46769
	4.32	63057
	6.04	79716
	7.94	96622
	10.00	113893
	12.21	131421
	14.58	149314
	17.03	167700
	19.70	185223
	22.45	205091

TABLE B.6
Rainstorm Summary

	<u>Duration</u>		
	12 hr. (mm)	24 hr. (mm)	72 hr. (mm)
a) <u>1:100 Year Return Period</u>			
1) Campbellton	85.6 (108.7)*	100.5 (126.3)*	N/A
2) Charlo Falls	N/A	106.7 (151.1)*	N/A
3) Campbellton Power	N/A	141.7 (196.9)*	N/A
4) Tide Head	N/A	202.6 (324.2)*	N/A
b) <u>Recorded Events*</u>			
1) Hurricane Belle	N/A	114.7**	190.5**
2) August, 1981	N/A	169.9**	207.9**
c) <u>Probable Maximum Rainfall</u>			
Campbellton		390	

* +95% confidence limits

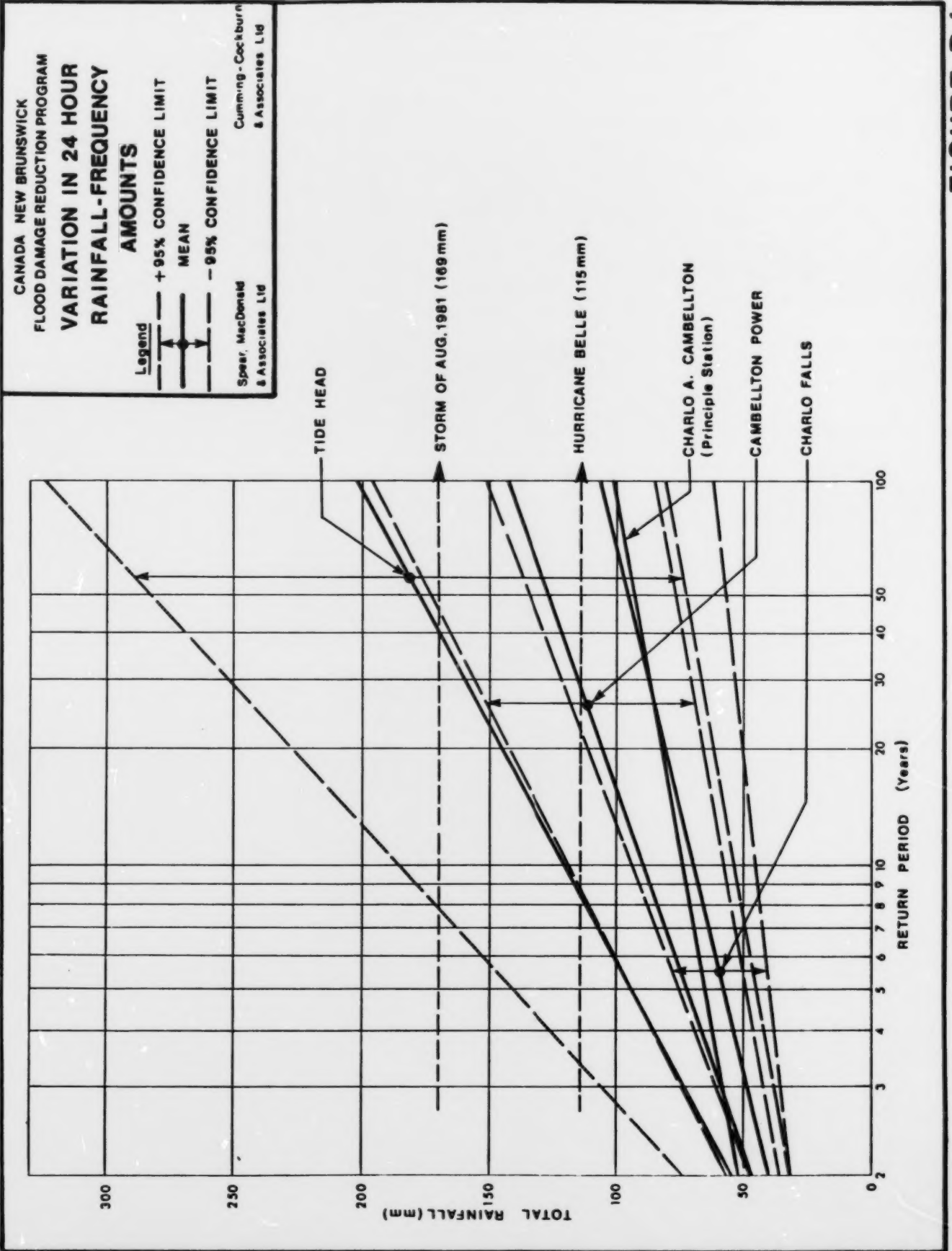
** maximum

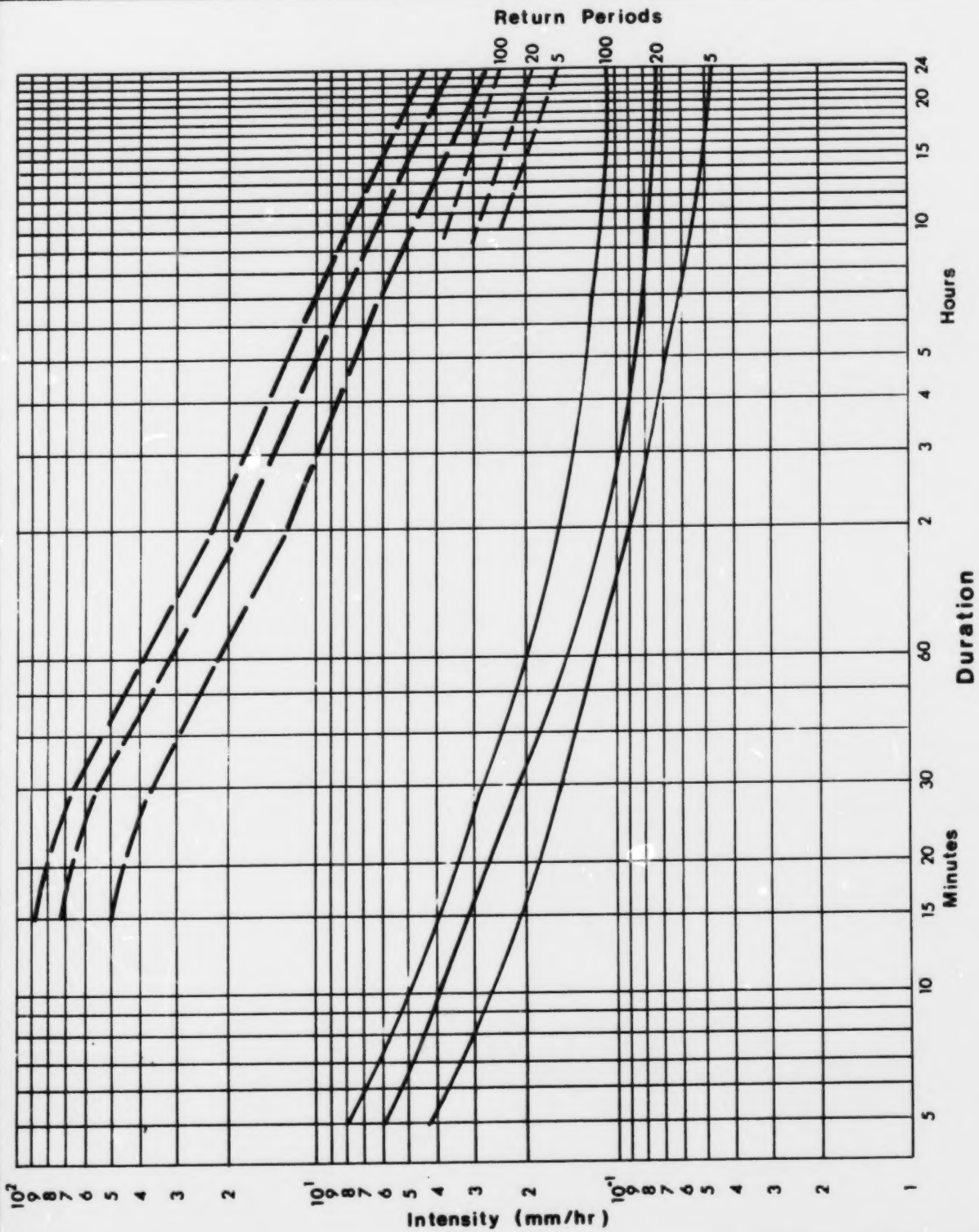
TABLE B.7

April/May Mean Temperature and Wind Velocities

	<u>Mean Daily Temp (°C)</u>	<u>Mean Wind Velocity (kph)</u>
April	2.5	13
May	9.1	13.9
Average	5.8	13.45

FIGURE B.1





- Rainfall (spring)
- - - Rainfall & Snowmelt (AES) (April/May)
- - - Rainfall (summer)

CANADA NEW BRUNSWICK
FLOOD DAMAGE REDUCTION PROGRAM
**RAINFALL INTENSITY DURATION
CURVES**

Spear, MacDonald
& Associates Ltd

Cumming - Cockburn
& Associates Ltd

